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Department of
Agriculture

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Conservation
Service

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April 8, 1982

TECHNICAL RELEASE NO. 68

SUBJECT: ENG — SEISMIC ANALYSIS OF RISERS

Purpose. To announce the distribution of Technical Release No. 68 — Seismic Analysis of Risers.

Effective Date. Effective when received.

Explanation. The technical release presents criteria and procedures for guidance in the analysis of drop inlet spillway risers subjected to earthquake loading. Both external stability and internal strength are considered.

Riser response to earthquake ground motion induces lateral forces. The lateral forces produce shears and moments with resulting concrete and reinforcing steel stresses.

Two environments are treated, these are the in-air and the in-water conditions. Further, the riser may be either free-standing or partially embedded in constructed fill.

Quantitative evaluation of the response of risers to seismic activity is new to SCS and hence the technical release deserves close attention.

Filing Instructions. File with other design criteria pertaining to drop inlet spillway risers.

Distribution. The technical release should be available to all SCS offices having responsibilities involving the planning and/or design of risers. Ten copies of TR-68 are distributed from National Headquarters to each State and NTC. Additional copies may be obtained from Central Supply.

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DIST: TR

TECHNICAL RELEASE
NUMBER 68

SEISMIC ANALYSIS OF RISERS

APRIL 1, 1982
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
NATIONAL ENGINEERING STAFF
DESIGN UNIT

PREFACE

This technical release presents criteria and procedure for the analysis of drop inlet spillway risers subjected to earthquake loading. An equivalent static analysis approach is employed to determine the effect of earthquakes on risers. Both external stability and internal strength are considered.

Lateral loads due to earthquakes are determined for two environments. These are the in-air and the in-water conditions. Further, the riser may be either free-standing or partially embedded in constructed fill.

This technical release is not a theoretical treatment of the dynamic response of risers to time-varying loads. It does contain some basic theory to develop concepts. Much of the material herein is approximate. It is hoped the technical release will stimulate thought and that improvements will be made with time.

A draft of the subject technical release dated July 1978 was circulated through Engineering and was sent to the Technical Service Center Design Engineers for their review and comment. Suggestions, comments, and questions by reviewers have helped the treatment herein.

The technical release was prepared by Edwin S. Alling, Head, Design Unit, National Engineering Staff, Glenn Dale, Maryland.

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SEISMIC ANALYSIS OF RISERS

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NOMENCLATURE

A_g ≡ gross area of concrete at section under investigation

A_s ≡ total steel area required in the wall at the section under investigation

B ≡ width of the riser parallel to the direction of motion, at the section under investigation

B_e ≡ effective width of the riser in the direction parallel to the applied forces, a "weighted" width

B_i ≡ width of segment i parallel to the direction of motion

C ≡ base shear coefficient; resultant compressive force on the wall under consideration

c ≡ a damping coefficient

D ≡ pipe conduit diameter

E ≡ modulus of elasticity

e ≡ eccentricity of resultant normal force on a section or bearing area

F_i ≡ lateral force acting on segment i

F_j ≡ lateral force acting on segment j

F_{je} ≡ force acting on endwall due to F_j

F_{js} ≡ lateral force acting on sidewall due to F_j

F_t ≡ lateral force assumed concentrated on the top of the riser

f_c ≡ concrete stress; allowable concrete stress

f_d ≡ a damping force

$f_e(t)$ ≡ time-varying externally applied force

$f_{eff}(t)$ ≡ effective support excitation loading

f_{min} ≡ minimum extreme fiber stress, used to determine if concrete section has cracked

f_s ≡ steel stress; allowable steel stress; the elastic or internal stiffness force in a dynamic analysis

f_t ≡ tensile stress in concrete

$f_f(t)$ ≡ an applied time-varying force

$f_w(t)$ ≡ resultant time-varying hydrodynamic force when riser is in-water

f_y ≡ steel yield stress

g ≡ acceleration of gravity

H_e ≡ depth of embedment of riser
H_h ≡ effective submerged height of riser
H_i ≡ distance from effective base of riser to c.g. of segment i
H_j ≡ height of segment j above effective base of riser

H_n ≡ effective height to normal water surface
H_r ≡ reduced, or effective depth of embedment of riser
H_s ≡ total effective height of riser
H_t ≡ total height of riser
H_w ≡ effective height of riser to crest of weir
H_x ≡ effective height of riser to level x
h_i ≡ length of segment i
I ≡ cross section moment of inertia
I_i ≡ moment of inertia of riser cross section at segment i
I_g ≡ gross moment of inertia of section neglecting reinforcing steel

i ≡ indexing number
J_o ≡ statical base moment reduction factor
J_x ≡ statical reduction factor for moment at level x
j ≡ indexing number
K ≡ ratio locating assumed axis of zero stress
K_a ≡ active lateral pressure ratio
K_p ≡ passive lateral pressure ratio
k ≡ stiffness term or spring constant for simple dynamic system
L ≡ width of the riser normal to the direction of motion
L_e ≡ weighted width of horizontal pressure diagram over effective embedment depth of riser

L_i ≡ width of segment i normal to the direction of motion
ℓ ≡ span length
M ≡ bending moment
M_o ≡ maximum overturning moment at base of riser
M'_o ≡ maximum overturning moment computed for riser of effective height H_s
M''_o ≡ overturning moment at base of embedded riser
$M_o(t)$ ≡ time-varying overturning moment at base of riser
M_s ≡ statical moment at level x
M_{sx} ≡ equivalent moment acting at location of resultant tensile force
M_w ≡ riser design wind moment

M_x ≡ moment at any level x

m ≡ mass of a particle; an upper limit of sequencing

m_a ≡ mass added to the particle to duplicate effect of water surrounding the riser

m_t ≡ total, or "virtual," mass of the particle

n ≡ the modular ratio, E_s/E_c; an upper limit of sequencing

p ≡ maximum bearing pressure under the riser

p_{max} ≡ maximum or limiting value of the resultant lateral earth pressure over the effective embedded depth of the riser

R ≡ structural response factor for tower structures

r ≡ ordinate of ellipse used to determine added mass values

S ≡ factor related to serviceability requirements and hazard class of the structure

SF_o ≡ safety factor against overturning

SF_s ≡ safety factor against sliding

T ≡ fundamental period of vibration of the riser in air; resultant tensile force at a section

T_{wf} ≡ fundamental period of vibration of the riser when fully submerged

T_{wp} ≡ fundamental period of vibration of the riser when partially submerged

t ≡ wall thickness of walls normal to the direction of motion; time

t_w ≡ wall thickness of walls parallel to the direction of motion

u ≡ relative displacement of the particle in the direction of the applied force

$\dot{u}$ = du/dt

$\ddot{u}$ = d^2u/dt^2

u_g ≡ displacement of the structure support

u_t ≡ total displacement of the particle

V_o ≡ total base shear

V'_o ≡ total base shear computed for riser of effective height, H_s

V''_o ≡ shear at the base of embedded riser

$V_o(t)$ ≡ time-varying base shear

V_x ≡ shear at any level x

v ≡ velocity of the particle, shear stress

v_c ≡ allowable shear stress, assumed carried by concrete

w_{ai} ≡ weight added to segment i to duplicate effect of water surrounding the riser

W_B ≡ buoyant weight of the riser

W_f ≡ buoyant or moist weight of the constructed fill over the riser footing

W_i ≡ weight of segment i

W''_o ≡ contact bearing reaction at base of embedded riser

W_T ≡ total effective weight of riser of effective height, H_s

W_{T_a} ≡ weight of the riser in air

W_x ≡ weight of the riser above any level x

w ≡ beam or riser weight per foot, unit pressure on riser walls

w_a ≡ weight per ft added to riser at height indicated by ellipse ordinate, r, to duplicate effect of water surrounding the riser

w_{ai} ≡ weight per ft added to segment i to duplicate effect of water surrounding the riser

w_i ≡ weight per ft of segment i

w_{js} ≡ lateral unit pressure on sidewall due to F_j

x ≡ effective height of particle above the base, an indexing number

Z ≡ seismicity zone factor

z ≡ abscissa of ellipse used to determine added mass values

z_i ≡ abscissa value for segment i

β ≡ ratio used to define ellipse from which added mass values are determined

β_e ≡ ratio used in determination of fundamental period of vibration of riser in water

β_i ≡ ratio used to define ellipse for segment i

γ ≡ unit weight of constructed fill

γ_b ≡ buoyant unit weight of constructed fill

γ_m ≡ moist unit weight of constructed fill

γ_w ≡ unit weight of water

δ_s ≡ statical deflection of the mass point of a single degree of freedom system

ζ ≡ abscissa of base of ellipse used to determine added mass values

ζ_i ≡ base abscissa used to determine added mass values for segment i

TECHNICAL RELEASE
NUMBER 68

SEISMIC ANALYSIS OF RISERS

INTRODUCTION

General Comment

The main purpose of this technical release is to consider the response of drop inlet spillway risers to earthquake ground motion and to formulate related analysis/design criteria. The location of such structures in an air/water environment is recognized.

That earthquakes can have a devastating effect on tower structures was evidenced during the February 9, 1971, San Fernando, California earthquake. This earthquake was assigned a Richter Magnitude of 6.6. The Lower San Fernando Dam had two outlet tower spillways. The outlet tower located near the center of the dam completely failed. It sheared off during the earthquake at about 20 feet above the base. The outlet tower near the west abutment sustained slight damage, but remained functional after the shaking. It should be observed that the tower near the center of the dam was subjected to forces transferred to the tower by reason of a mass slide in the upstream face of the embankment. Forces generated by incipient slope failure are beyond the scope of this technical release. See discussion of partially embedded risers.

The response of risers in air is presented prior to the treatment of risers surrounded by water. Criteria and procedures are given for the determination of riser design adequacy considering earthquake risk and intensity and also considering structure hazard class.

The equivalent static analysis method presented herein is frankly approximate. It is an attempt to recognize, with a simple procedure, the effect of earthquakes on risers. The material presented herein is meant to provide guidance. The designer may determine it advisable to adjust various suggested investigations, procedures, and/or parameter values in the light of his experience and judgement. It is expected that this technical release will be revised with time as knowledge is gained and refined methods become more readily adaptable.

Initial Development

Initial discussions and development of concepts are presented without regard for foundation influences. Several tacit assumptions are thus inherent in the resulting expression for total base shear. To be strictly applicable, the riser is fixed to bedrock. Thus the riser base and bedrock are subjected to identical earthquake ground motion.

Foundation influence adjustments for actual site conditions and soil-structure interaction are presented subsequently. It is of interest that the total base shear and concomitant function values neglecting foundation influence adjustments are often sufficiently accurate.

RISERS IN AIR

General Comment

This section begins with a theoretical development of the equations of motion of structures. The presentation is an extremely brief, simplified introduction to the general topic of structural vibrations. The formulation is provided for the insight it may furnish to the understanding of the dynamic response of risers to time-varying loadings.

The response of a structure to dynamic loading depends on the definition of the loading, the resistance of the structure to deformation, and the mass distribution of the structure. Further, if the structure's supports are not immovable, their motion must also be defined in terms of time.

To analyze the dynamic behavior of a structure, one must be able to define its position at any instant. If this is possible, then not only can the deformation of the structure from its reference position be determined, but also it is possible to compute the way strains, and hence stresses, vary with time.

If, at any instant, the position of a structure can be defined by one number, or one coordinate, the structure is said to have one degree of freedom. The mass of an actual structure is continuously distributed over the structure. As a consequence of this distributed mass, actual structures have an infinite number of degrees of freedom. This is because the definition of the structure's position requires the specification of the displacement of every point of the structure, of which there are an infinite number.

Often the important features of the dynamic response may be adequately approximated by idealizing the structure. In such idealizations, the mass of the structure is considered to be lumped or concentrated at certain discrete mass points. The resistance of the structure to deformation is then represented by elements which are considered weightless, but which have structural strength and stiffness. Theoretical studies then often proceed with a study limited to the first few modes of vibration.

Theoretical Development

The theoretical development of the equations of motion is presented in this section in terms of a single particle of mass. The response of the total structure to a dynamic loading would be obtained by summation over the entire structure or over the assumed lumped-mass system.

Formulation of the equations of motion.--The equations of motion of a dynamic system may be obtained from Newton's second law of motion. This law states that the time rate of change of momentum of a mass is equal to the force acting on the mass. The mathematical relation in differential form is:

$$f_t(t) = \frac{d(mv)}{dt} = \frac{d}{dt}\left(m \frac{du}{dt}\right) \tag{1}$$

where:

$f_t(t) \equiv$ the applied time-varying force
$m \equiv$ the mass of the particle
$v \equiv$ the velocity of the particle
$u \equiv$ the displacement in the direction of the applied force.

For present purposes mass remains constant, thus:

$$f_t(t) = m \frac{d^2u}{dt^2} \equiv m\ddot{u} \tag{2}$$

This may be written as:

$$f_t(t) - m\ddot{u} = 0 \tag{3}$$

which is an illustration of the application of d'Alembert's principle. The second term, $m\ddot{u}$, is the inertia force resisting the acceleration of the mass.

d'Alembert's principle permits treatment of dynamic problems as problems in static equilibrium. The inertia force is applied with a sense opposite to the acceleration. The force $f_t(t)$ includes all loads and load components acting on the mass in the direction paralleling the acceleration. The force may thus include externally applied loads, damping forces which are functions of velocity, elastic constraints which are functions of displacement, and so on.

Thus let:

$$f_t(t) = f_e(t) - f_d - f_s \tag{4}$$

where:

$f_e(t) \equiv$ time-varying externally applied force

$f_d = c\dot{u} \equiv$ the damping force which is the product of a damping coefficient and the velocity

$f_s = ku \equiv$ the elastic force which is the product of a stiffness term and the displacement

Therefore from:

$$f_t(t) - m\ddot{u} = 0 \tag{3}$$

then:

$$m\ddot{u} + c\dot{u} + ku = f_e(t) \tag{5}$$

which is the basic equation of motion for a single degree of freedom system with an externally applied force.

Influence of supports.--Dynamic stresses, strains, and displacements may be caused in a structure not only by externally applied time-varying loads, but also by motions of the structure's supports. This is the case with buildings or towers in the presence of an earthquake. Accelerations of elements of the structure are produced by the movements of the structure foundation. Since there is no externally applied load:

$$f_t(t) = -f_d - f_s \tag{6}$$

The inertia force depends on the total displacement of the mass from a reference point, while the damping and elastic forces are functions of relative displacements. Thus:

$$m\ddot{u}_t + c\dot{u} + ku = 0 \tag{7}$$

where:

$u_t \equiv$ total displacement measured from a reference axis.

This is the basic equation of motion for a single degree of freedom system with earthquake induced movement.

Risers - excitation by rigid-base translation.--Vertical components of earthquake movements are excluded. All support points are subjected to identical horizontal translations. Unidirectional horizontal translations are considered. Hence, working with a particle of the tower structure at height x above the base, see figure 1, the basic equation of motion is again:

$$m\ddot{u}_t + c\dot{u} + ku = 0 \tag{7}$$

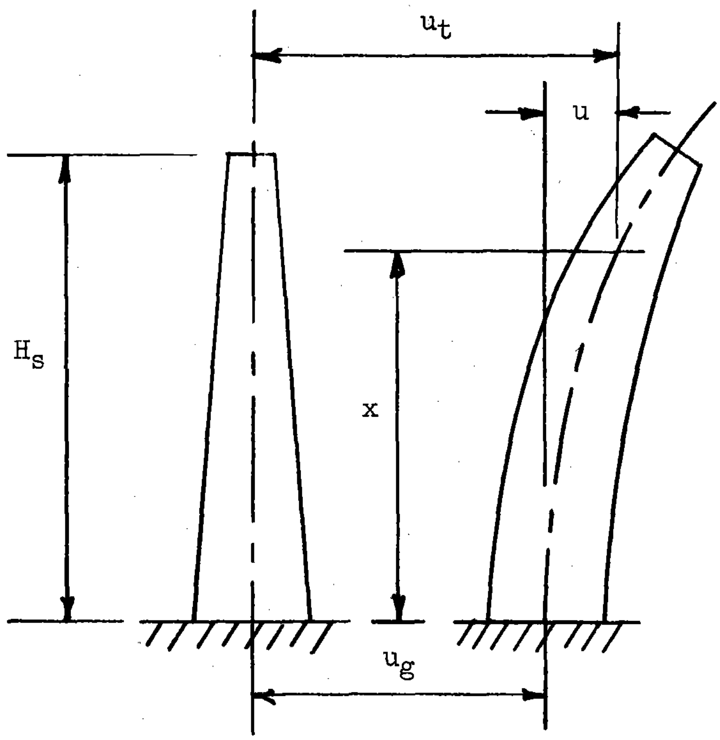


Figure 1. Tower structure with rigid-base translation.

To solve this equation, all forces must be given as functions of a single variable. The total displacement of the particle can be written as the sum of the ground translation and the translation of the particle due to structure distortion, or:

$$u_t = u_g + u \tag{8}$$

where:

$u_t \equiv$ total displacement of the particle
$u_g \equiv$ displacement of the support
$u \equiv$ relative displacement of the particle

so:

$$m\ddot{u} + m\ddot{u}_g + c\dot{u} + ku = 0 \tag{9}$$

or:

$$m\ddot{u} + c\dot{u} + ku = - m\ddot{u}_g \tag{10}$$

This is often written as:

$$m\ddot{u} + c\dot{u} + ku = f_{eff}(t) \tag{11}$$

where:

$$f_{eff}(t) \equiv - m\ddot{u}_g \tag{12}$$

Thus the tower particle responds to the ground acceleration, $\ddot{u}_g$, exactly as it would to an externally applied time-varying load equal to the product of the mass of the particle and the ground acceleration. The term, $f_{eff}(t)$, is sometimes called the effective support excitation loading. This loading, see figure 2, is shown as varying over the height of the structure, H_s. As indicated in the figure, resisting base shear, $V_o(t)$, and base moment, $M_o(t)$, are induced by this variable loading.

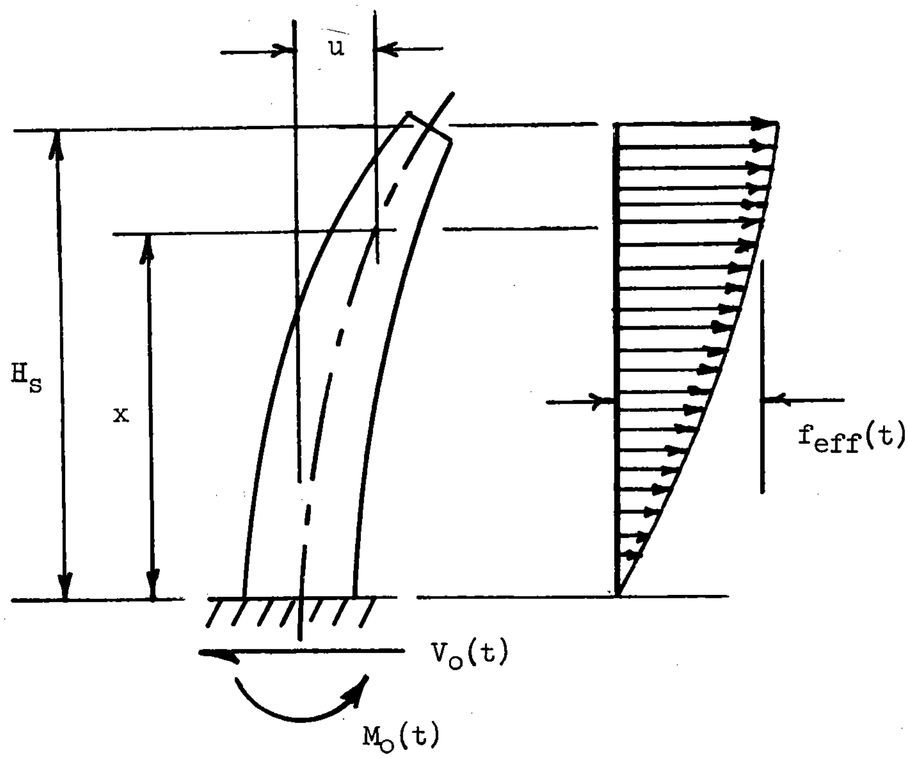


Figure 2. Effective support excitation loading.

Complexities and uncertainties.--The equation of motion just developed, equation 11, was obtained essentially by consideration of a single-particle of mass in a structure. Also, the structure was constrained to a single degree of freedom; that is, a system for which the displacement can be represented by the amplitude of a single coordinate.

Although the complete response of some multi-degree of freedom systems subjected to general earthquake motions can be calculated, it is not generally necessary to make such computations as a routine matter in the design of risers. Such solutions are impractical without the aid of computers. Further, there are a number of questions that must be resolved in order to proceed theoretically. Some of these are concerned with: the configuration of the structure, the material properties of the structure and the geological and seismological conditions surrounding the site - including the prediction of the location and magnitude of possible earthquakes in the region along with the time-sequence of horizontal ground motions at the site. The consequences of failure or the hazard class of the structure are also important considerations.

In view of these complexities and uncertainties, a pseudo-static analysis is adopted in lieu of requiring a more difficult dynamic analysis.

The basic equation of motion for rigid-base translations, equation 11, will be considered subsequently for the insight it provides to the case of risers surrounded by water.

Equivalent Static Analyses

Basis.--The pseudo-static model for seismic analyses of risers, contained herein, results primarily from a blending of the procedures from three sources. These are: (1) Uniform Building Code, International Conference of Building Officials; (2) "Specifications for the Design and Construction of Reinforced Concrete Chimneys," ACI (307-69); and (3) "Tentative Provisions for the Development of Seismic Regulations for Buildings," Applied Technology Council, Structural Engineers Association of California.

Concepts.-- The method includes the use of lateral force coefficients corresponding to relative accelerations varying linearly from zero at the base to a maximum at the top of the structure. This distribution over the height of the structure takes into account the fact that acceleration of higher levels of the tower are greater than those at lower levels due to the greater displacements that occur at the higher elevations.

However, strict adherence to a linear variation in acceleration causes maximum shears in the higher elevations to be underestimated. The need for higher shears near the top of the tower is met by assuming a portion of the base shear is caused by a concentrated force applied at the top of the riser, while the remainder is distributed in accordance with the linear variation of horizontal accelerations.

A covered top riser is used for illustration in the following development. It is also used in the subsequent example computations. Of course, any type riser can be accommodated. The figures that follow imply an earthquake response about an axis parallel to the riser sidewalls; that is, motion is parallel to the planes of the endwalls and the generated forces are normal to the sidewalls. The relations, suitably interpreted, also apply to response about an axis parallel to the endwalls.

The equivalent static analysis method furnishes a procedure for determining the maximum lateral forces, the maximum shear, and the maximum moment at any level in the riser due to the assumed earthquake conditions.

Total base shear.--The total base shear, V_o, acting in a direction parallel to one of the principal horizontal axes of the riser, is determined (but see also equation 53 and associated discussion) as:

$$V_o = ZSRCW_T \tag{13}$$

where:

- $Z \equiv$ zone factor, see seismic zone map, for example that contained in TR-60. For zone 0, $Z = 0.00$; for zone 1, $Z = 0.25$; for zone 2, $Z = 0.50$; for zone 3, $Z = 1.00$; and for zone 4, $Z = 1.33$
- $S \equiv$ factor related to importance or serviceability requirements and hazard class of the structure. For class (a) structures, $S = 1.00$; for class (b), $S = 1.50$; and for class (c), $S = 2.00$
- $R \equiv$ structure response factor, $R = 2.00$ for risers, towers, and chimneys
- $C \equiv$ base shear coefficient, see next section for evaluation
- $W_T \equiv$ total effective weight of structure.

Base shear coefficient.--The base shear coefficient is taken as a function of the fundamental period of vibration of the riser. With the period, T, in seconds, the coefficient is given by:

$$C = 0.05/T^{1/3} \tag{14}$$

however C need not be taken greater than 0.10.

Fundamental period of vibration.--The fundamental period of vibration of the riser in the direction under consideration is required. It may be determined as:

$$T = 2\pi \sqrt{\frac{\delta_s}{g}} \qquad (15)$$

where δ_s is the statical deflection of the mass point of a single degree of freedom system. By this approach the period for a prismatic cantilever beam fixed at the base is approximately:

$$T \simeq \frac{2\pi}{3.567} \sqrt{\frac{wH_s^4}{EIg}} = \frac{2\pi}{3.567} \sqrt{\frac{W_T H_s^3}{EIg}} \qquad (16)$$

where, using consistent units:

$w \equiv$ weight per ft of the beam, plf

$H_s \equiv$ total effective height of riser, ft

$E \equiv$ modulus of elasticity, psf

$I \equiv$ moment of inertia of the beam cross section, ft^4

$g \equiv$ acceleration of gravity, ft per sec^2

Following similar reasoning, the period for a nonprismatic cantilever beam, that is, riser, can be found approximately as:

$$T \simeq \frac{2\pi}{3.567} \sqrt{\frac{3W_T}{Eg} \sum_{i=1}^{m} (H_w - H_i)^2 \frac{h_i}{I_i}} \qquad (17)$$

where, referring to figure 3:

$H_w \equiv$ height of riser to crest of weir, ft

$H_i \equiv$ distance from base of riser to c.g. of segment i, ft

$h_i \equiv$ length of segment i, ft

$I_i \equiv$ moment of inertia of riser cross section at segment i, ft^4

Note that the summation process used here extends from the base of the riser to the weir crest.

The value of T obtained from equation 17 should be used except that it should not be taken larger than:

$$T = 0.05\, H_s/B_e^{1/2} \qquad (18)$$

where:

$B_e \equiv$ effective width of the riser in a direction parallel to the applied forces; B_e may be determined as a "weighted" width of the riser including the footing but excluding the cover slab and cover slab walls, ft (see example).

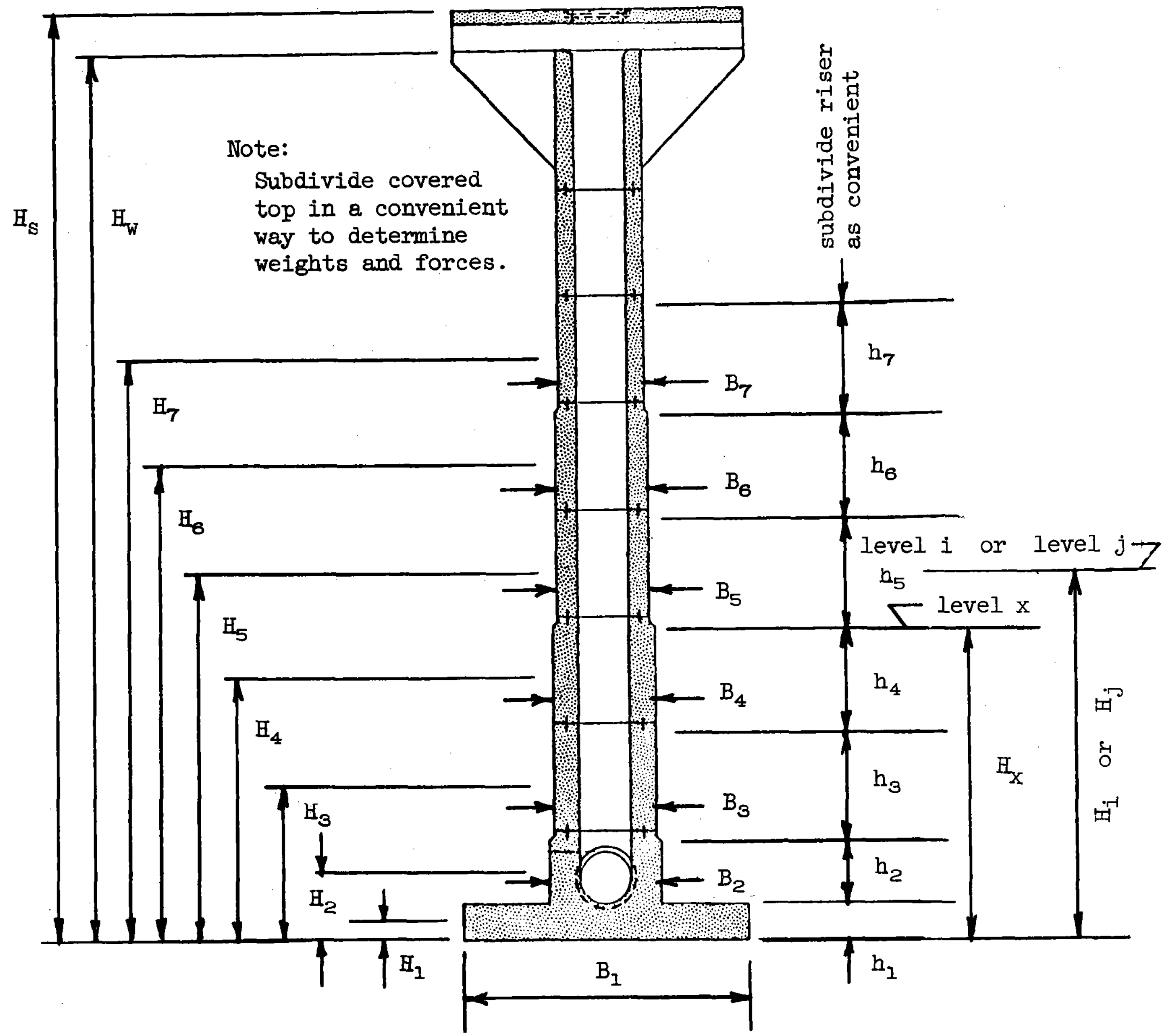


Figure 3. Definition sketch - forces normal to sidewall.

<u>Weight of structure</u>.--The total effective weight of the structure may be found as:

$$W_T = \sum_{i=1}^{n} W_i \tag{19}$$

with: $W_i = w_i' h_i$ (20)

where: W_i = weight of segment i, lbs

Note that the summation process used here includes the entire structure.

Top lateral force.--The assumed concentrated force applied at the top of the riser is given by:

$$F_t = 0.004\left(\frac{H_s}{B_e}\right)^2 V_o \qquad (21)$$

however F_t should not be taken greater than $0.15V_o$.

Distribution of remaining lateral forces.--The lateral force, F_j, acting on each segment, j, including the uppermost level, n, is given by:

$$F_j = (V_o - F_t)\frac{W_jH_j}{\sum_{1=1}^{n} W_iH_i} \qquad (22)$$

The force may be treated as concentrated at the c.g. of the segment; or whenever desirable to do so, it may be assumed distributed over the length, h_j, of the segment. Figure 4 portrays the variation of lateral forces on the riser. The force due to the covered top, may be determined as one or more F_j forces by subdividing the top in any convenient manner to obtain weights and c.g. locations.

Shears.--The shear at any level x is determined by summing the lateral forces from the top of the riser to the level under consideration, thus:

$$V_x = F_t + \sum_{j=x}^{n} F_j \qquad (23)$$

The shear at the base can be determined as:

$$V_o = F_t + \sum_{j=1}^{n} F_j \qquad (24)$$

which should check the previously given:

$$V_o = ZSRCW_T \qquad (13)$$

Moments.--Overturning moments computed from more theoretical work are found to be smaller than values computed as the statical moment of the lateral forces described above. The reason for this difference lies in the fact that the given distribution of lateral forces is patterned to duplicate the maximum shear values occurring over the height of the structure. That is, maximum shear values do not occur simultaneously at all elevations. Hence the area under the maximum shear envelope exceeds the actual demands on resistance to overturning moments.

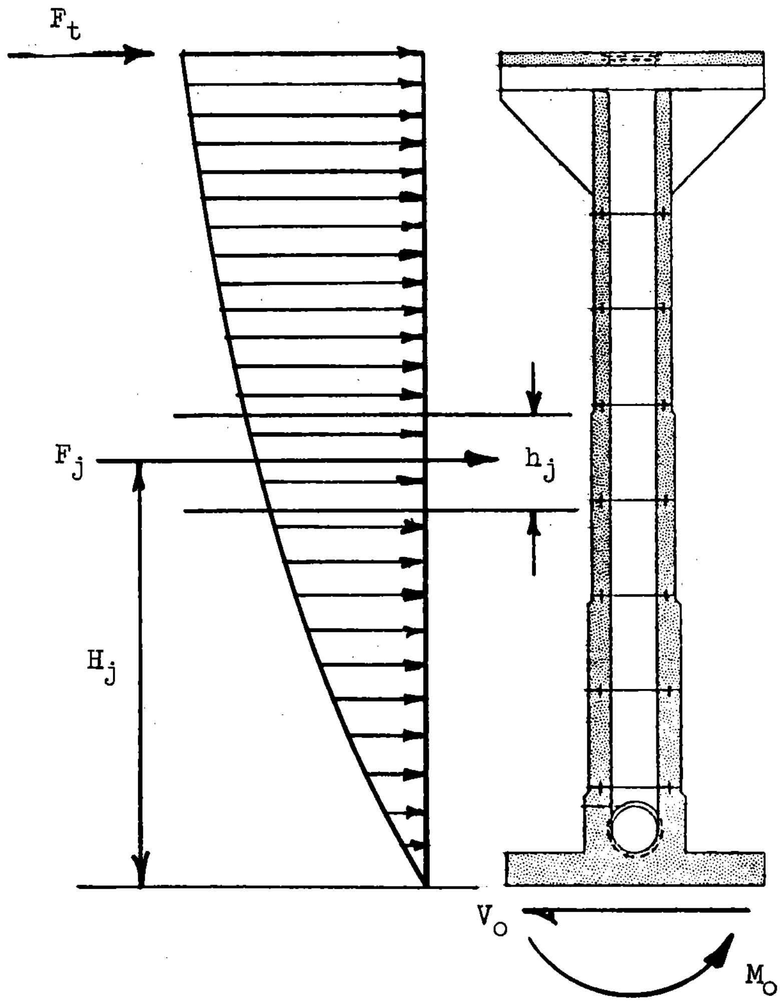


Figure 4. Lateral forces on riser.

This effect is recognized by applying a reduction factor to the computed statical moments of the lateral forces. Thus, the overturning moment about the base of the riser is given by:

$$M_o = J_o \left\{ F_t H_s + \sum_{j=1}^{n} F_j H_j \right\} \tag{25}$$

where $J_o \equiv$ statical base moment reduction factor.

Take:

$$J_o = 0.6/T^{1/3} \tag{26}$$

but J_o should not be taken less than 0.80 nor more than 1.00.

Similarly, the moment at any level x is given by:

$$M_x = J_x \left\{ F_t (H_s - H_x) + \sum_{j=x}^{n} F_j (H_j - H_x) \right\} \tag{27}$$

where $J_x \equiv$ statical moment reduction factor for moment at level x.

Take:

$$J_x = J_o + (1. - J_o)\left(\frac{H_x}{H_s}\right)^3 \tag{28}$$

RISERS IN WATER

The dynamic response of a drop inlet spillway riser to earthquake ground motion is increased when the riser is surrounded by water over what it is when the riser is in air. The question to be considered is thus how to account for the changed behavior of the riser due to the surrounding water.

General Comment

The increase in the dynamic response of risers to earthquake motion when water surrounds the riser can be expected to range from slight to very significant. Comparisons, using complex theoretical analyses, of stresses in towers surrounded by water versus towers in air show that maximum bending stresses will sometimes more than double and that maximum shearing stresses will sometimes more than triple.

A satisfactory, relatively simple, procedure to account for the effects of the riser-water system interaction during seismic activity is needed. The riser may be either fully or partially submerged. Water inside the riser presents no particular problem. This water is treated as moving rigidly with the riser and hence should be included as part of the actual mass of the riser in all calculations. This is an adequate idealization since, where earthquake effects are significant, the inside plan dimensions of the riser are usually much smaller than the depth of water.

Early investigations of water pressures on dams during earthquakes led to two alternative concepts for visualizing the effects of water against the face of a dam. The first concept treats the effect of earthquake motion in terms of creating added (in excess of hydrostatic) water pressures against the dam. Shears and moments can then be determined from the pressure distribution. The second concept treats the effects of earthquake motion by assigning, or adding, extra mass to the structure. In terms of riser analyses, this added mass, when correct in magnitude and distribution, will allow the riser to be investigated essentially the same as a riser in air.

While the added water pressure concept is easily applied to the face of a dam, it does not lend itself to accommodation of tower structures whose dimensions vary with height. The added mass concept is thus preferred since treatment can duplicate the analysis of a riser in air once the added mass is determined.

Effect on Response Behavior

Riser interaction with surrounding water is found to have the following effects. The fundamental period of vibration of a tower structure is increased. Damping of tower vibrations is decreased. Displacements are increased. Overall, these changes lead to increased effective lateral forces on risers in water.

Theoretical Development for Riser-Water System

Water surrounding cantilever tower structures causes additional dynamic forces and modifications in the dynamic properties of these structures. The solution formulation must therefore include the hydrodynamic interaction between the riser and the surrounding water.

Formulation of equation of motion.--The basic equation for rigid-base translation of tower structures in air was found to be:

$$m\ddot{u} + c\dot{u} + ku = f_{eff}(t) \tag{11}$$

To this expression must be added a term to account for the hydrodynamic (in excess of hydrostatic) pressures acting on the outside of the riser at height x above the base, or:

$$m\ddot{u} + c\dot{u} + ku = f_{eff}(t) + f_w(t) \tag{29}$$

where:

$f_w(t) \equiv$ the resultant time-varying hydrodynamic force per unit of height in the direction of motion caused by the rigid-base translation.

Treatment of the force, $f_w(t)$, requires consideration of the pressure effects of normal, tangential, and vertical components of water particle displacement. Sophisticated methods of solution, including finite element analyses, are available and have been applied to various simplified types of tower structures. Such advanced work shows that compressibility of water is of essentially no consequence for these structures and that the effects of surface waves can also be ignored. As before, attention herein is turned toward easily applied pseudo-static procedures.

Added water pressures.--Equation 29 leads to the concept of determining added water pressures acting on the riser as a result of horizontal movements of the riser in the water. These added water pressures could be treated as loadings in equivalent static analyses. This approach is not pursued further herein for seismic analyses of risers, although the concept does lend itself well to the investigation of earthquake effects on gravity dams.

Added mass.--An alternative formulation to determine hydrodynamic pressures, due to water surrounding the riser, is possible. The thought process is the reverse of that used earlier. Before, the substitution of an externally applied time-varying load, $f_{eff}(t)$, for the inertia force due to displacement of the structures support, $-m\ddot{u}_g$, was made. That is:

$$f_{eff}(t) = -m\ddot{u}_g \tag{12}$$

was used in obtaining equation 11. Now, the concept is to replace the hydrodynamic force term with an equivalent inertia force, $-m_a\ddot{u}$, or let:

$$-m_a\ddot{u} = f_w(t) \tag{30}$$

so that from equations 29 and 30:

$$(m + m_a)\ \ddot{u} + c\dot{u} + ku = f_{eff}(t) \tag{31}$$

or:

$$m_t\ddot{u} + c\dot{u} + ku = f_{eff}(t) \tag{32}$$

where:

$m_a \equiv$ mass added to the particle at height x above the base; the mass must be just sufficient to produce the equality expressed in equation 30.

$m_t = m + m_a \equiv$ total, or "virtual," mass of the particle.

Restating the concept; one may visualize a certain body of water rigidly connected to, and moving with, the riser while the remainder of the surrounding water remains inactive and unassociated with the riser. The shape of the mass of water considered to move with the riser must be selected so that the inertia forces become equal to the excess pressures actually exerted by the water on the riser due to the dynamic action.

By this concept, the effects of the surrounding water on the dynamics of a riser are represented by an added mass distributed along the height. The total (or virtual) mass of the riser is the sum of the mass of the riser itself (including any water inside the riser) and the added mass.

This virtual mass is then used in a standard equivalent static analysis just as if the tower were not submerged in water.

Figure 5 shows a tower structure partially submerged in water and subjected to unidirectional horizontal ground translation. The distributed mass to be added to the system over the submerged height, H_h, is indicated.

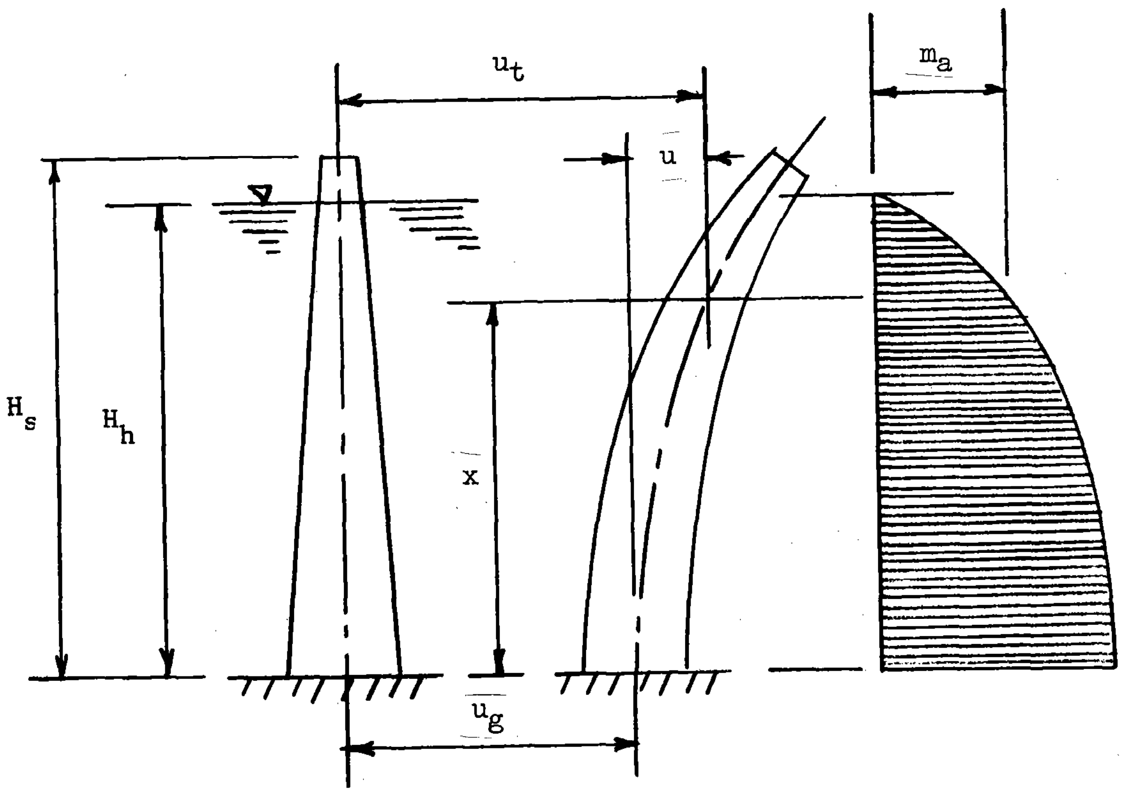


Figure 5. Tower structure partially submerged in water, subjected to rigid-base translation, indication of added mass.

Equivalent Static Analyses - Added Mass Approach

The first step in the investigation of a riser surrounded by water is the determination of the distributed added mass to be combined with the mass or weight of the riser plus water inside the riser.

Determination of added mass.--For a riser or tower of constant section, the added mass at any point above the base of the riser, within the height, H_h, is defined by the expression for the quadrant of an ellipse, see figure 6.

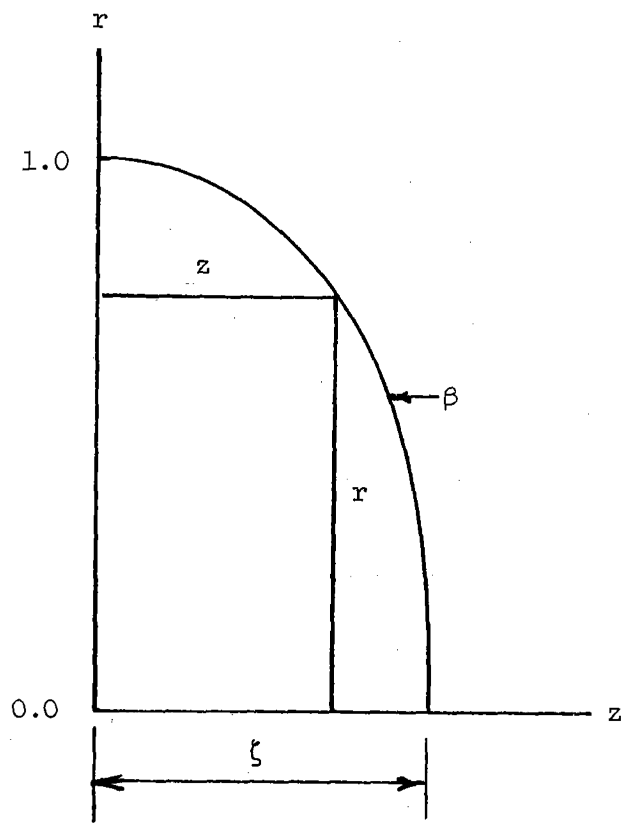


Figure 6. Ellipse for determination of added mass.

The relation for the indicated β ellipse is:

$$r^2 + \frac{z^2}{\zeta^2} = 1 \tag{33}$$

or:

$$z = \zeta \sqrt{1 - r^2} \tag{34}$$

For the ellipse, the coordinates r and z are dimensionless and are:

$$r = \frac{H_i}{H_h} \tag{35}$$

and:

$$z = \frac{w_a}{\gamma_w H_h L} \tag{36}$$

where:

w_a ≡ the added weight per ft of height at the point above the base indicated by the ratio r, plf

γ_w = 62.4 ≡ unit weight of water, pcf

L ≡ width of the riser, normal to the direction of motion, ft

The abscissa of the base of the ellipse, ζ, is a measure of the stiffness of the riser and is given by:

$$\zeta = 0.8\beta - 0.2\beta^2 \tag{37}$$

when $\beta \leq 2$, and by:

$$\zeta = \frac{14\beta}{3 + 16\beta} \tag{38}$$

when $\beta > 2$, where:

$$\beta = B/H_s \tag{39}$$

note that:

$B \equiv$ width of the riser, parallel to the direction of motion, ft.

In the usual case the cross section of the riser varies with height above the base. It is recommended that the added weight for any segment i of a riser with variable wall thickness be determined from the relations pertaining to the riser section at segment i. Thus:

$$w_{ai} = z_i \gamma_w H_h L_i \tag{40}$$

where:

$$z_i = \zeta_i \sqrt{1 - r^2} \tag{41}$$

in which, if $\beta_i \leq 2$:

$$\zeta_i = 0.8\beta_i - 0.2\beta_i^{\,2} \tag{42}$$

and, if $\beta_i > 2$:

$$\zeta_i = \frac{14\beta_i}{3 + 16\beta_i} \tag{43}$$

where:

$$\beta_i = B_i/H_s \tag{44}$$

In the region of the riser top, neglect cover slab and cover slab walls in evaluating B_i, that is, use out-to-out of riser walls.

Finally then, the weight added to segment i becomes, in lbs:

$$W_{ai} = w_{ai} h_i \tag{45}$$

Determination of fundamental period if vibration.--As noted earlier, the fundamental period of a riser surrounded by water is greater than that of the riser in air.

The value of the fundamental period of a fully submerged riser, T_{wf}, can be determined, in seconds, in terms of the period in air, T, as:

$$T_{wf} = (1.46 - 0.77\beta_e + 0.70\beta_e^2)T \tag{46}$$

when $\beta_e \leq 0.5$, and by:

$$T_{wf} = (1.285 - 0.07\beta_e)T \quad (47)$$

when $\beta_e > 0.5$ where:

$$\beta_e = \frac{B_e}{H_s} \quad (48)$$

The value of the fundamental period of a partially submerged riser, T_{wp}, can be determined, in seconds, in terms of the period of air, T, and the fully submerged period, T_{wf}, as:

$$T_{wp} = T + (9/4)(H_h/H_s - 1/3)^2 (T_{wf} - T) \quad (49)$$

but T_{wp} shall not be taken less than T.

Determination of effective lateral forces.--With T_{wf} or T_{wp} known, and with the weights to be added to each segment known, the analysis now proceeds as for a riser in air. The total base shear, V_o, is determined from equation 13. The base shear coefficient, C, is determined from equation 13 using the appropriate period. The top lateral force, F_t, is determined from equation 21. The remaining lateral forces are determined from equation 22. Then shears and moments are determined from equations 23 through 28.

RISERS PARTIALLY EMBEDDED IN CONSTRUCTED FILL

General Comment

Risers located in the reservoir are often founded essentially at original ground level as free-standing structures. Thus during the early portion of project life, the total height of the riser would be exposed during an earthquake. The response of a riser to earthquake ground motion is related to the total effective height of the riser. If the total effective height can be reduced, then the riser response will also be reduced.

Embedment of the riser in constructed fill reduces the total effective height and hence is beneficial. Embedment is particularly desirable in those situations for which the riser should be analyzed for in-water conditions. The amount of embedment possible or practical will be dictated by economics or by project or site requirements; for example, dry dam or elevation of low stage inlet opening. Embedment as used herein means surrounded, that is, encased in constructed fill. It does not mean surrounded by the sediment that accumulates over the life of the structure. The constructed fill may be a part, or logical extension, of the earth dam embankment, or it may be a local fill in the region of the riser. A considerable earth mass is required if the embedment is to be effective in reducing riser response to ground motion. As a minimum, the surface of the embedment should probably extend outward from the riser in all directions a distance of at least twice the depth of embankment.

Embedding a riser in an embankment should not be expected to prevent a sliding failure of the embankment. That is, the embankment should be stable unto itself during a seismic event. There should be no presumption that the riser can serve as a pin or dowel preventing soil movement along a potential slip surface. The analyses which follow treat only the response of an embedded riser to earthquake loading.

Effect of Embedment

Partially embedding a riser in constructed fill has two effects on the response of the riser to earthquake shock. The total effective height of the riser is reduced. The overturning moment at the base of the riser is reduced from what it would be without the embedment. Quantification of these effects is quite inexact.

Total effective height.--The total effective height, H_s, to be used in the earthquake analyses must be determined. The height should not be taken as the total riser height minus the depth of embedment. Rather, it should be something more than this amount. This is true because repeated cycles of earthquake vibrations will eliminate or significantly reduce the resistance the upper portion of the embankment can offer to the shaking. Therefore, referring to figure 7, the total effective height of the riser for determination of earthquake lateral loads, shears, and moments is taken as the smaller of:

$$H_s = H_t - \frac{2}{3} H_e \tag{50}$$

or:

$$H_s = 2(H_t - H_e) \tag{51}$$

where:

$H_t \equiv$ total height of riser, ft

$H_e \equiv$ depth of embedment, ft.

This value of H_s discounts any help of the upper portion (upper third, or distance equal to the exposed height) of the embedment in establishing the stability of the riser. With H_s known, the analysis then proceeds as if the riser had an actual height of H_s. The riser below the effective H_s is considered non-existent in computing the riser response over the height H_s.

Force system on effective embedded portion.--Embedment of the riser reduces the overturning moment that must be resisted at the base. The procedure indicated below, by which the base shear and the reduced base moment are evaluated, is approximate. It is somewhat similar to concepts used in analyzing embedded cantilever sheet pile walls subjected to lateral loads. Figure 8 illustrates significant features of the analysis.

Sketch (a) shows the overturning moment, M_o', and base shear, V_o', computed for a riser of effective height, H_s. The sketch also shows the reduced base moment, M_o'', and base shear, V_o'', acting at the base of the riser. The effective embedment depth, H_r, is also indicated. The embedded portion of the riser rotates about a pivot point and thus bears against the effective embedment fill. Sketch (b) shows an assumed pivot point at the

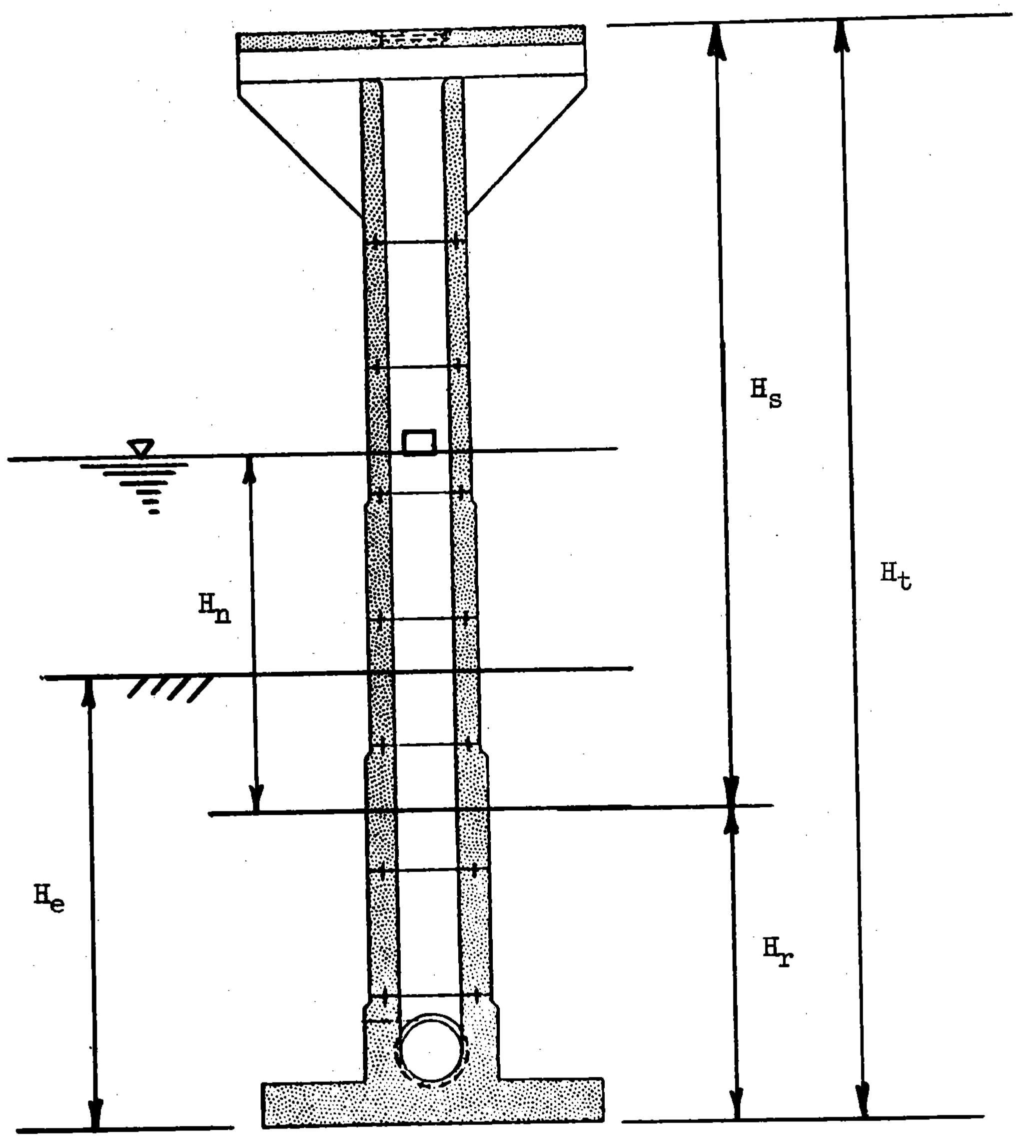


Figure 7. Partially embedded riser, determination of effective H_s.

bottom of the riser. In the analysis of embedded flagpoles or isolated piles, the pivot point is usually located some distance up from the bottom of the pole. This location is generated by equilibrium requirements with zero moment at the bottom of the pole. In the case of embedded risers, equilibrium is satisfied as indicated by the free body diagram of sketch (d).

Displacement of the riser against the fill causes a curvilinear distribution of resultant lateral earth pressures. For convenience the distribution is taken as linear with the maximum pressure occurring at mid-depth as shown in sketch (c). On the upper half of the effective embedment depth, the maximum resistance that can be developed is taken as the difference between passive pressure on the displacement side of the riser and active pressure on the opposite side. On the lower half, the maximum resistance is assumed, because of the pivot point, to decrease linearly to zero from its value at mid-depth. Hence, the limiting value of p_{max} shown in sketches (c) and (d) is, in psf:

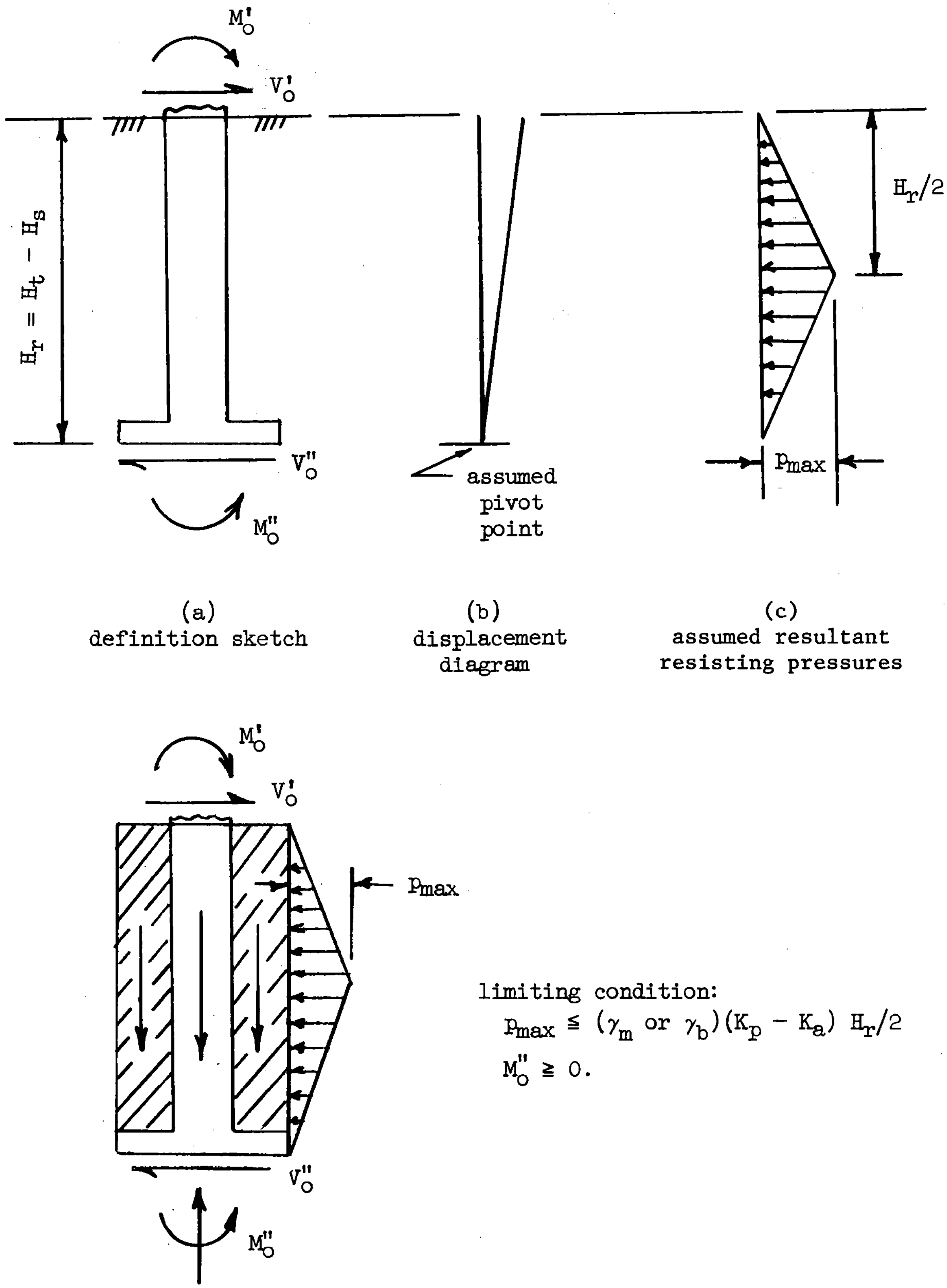
$$p_{max} = (K_p - K_a)\ \gamma\ H_r/2 \tag{52}$$

in which γ equals γ_m or γ_b as appropriate to the analysis, and where:

K_p ≡ passive lateral earth pressure ratio
K_a ≡ active lateral earth pressure ratio
γ ≡ unit weight of fill, pcf
γ_m ≡ moist unit weight of fill, pcf
γ_b ≡ bouyant unit weight of fill, pcf.

The value of p_{max} that will make the reduced base moment, M''_o, equal zero is determined from statics. If p_{max}, so computed, exceeds its limiting value from equation 52, then p_{max} is set at its limiting value and the resulting M''_o is determined by statics. The base shear, V''_o, then follows directly from statics.

A brief might be presented for two possible variations in evaluating the limiting value of p_{max} as given by equation 52 above. First, it might be argued that the embedded portion of the riser may displace far enough and fast enough to possibly lose contact with the fill. If this occurs, the passive pressures on the displacement side of the riser would act without simultaneous active pressures on the opposite side of the riser. Second, it might be argued that only in very coarse fills could pore pressure



limiting condition:

$$p_{max} \leq (\gamma_m \text{ or } \gamma_b)(K_p - K_a)\, H_r/2$$

$$M_o'' \geq 0.$$

(d)
free body diagram

Figure 8. Analysis of effective embedded portion of riser.

dissipation possibly be rapid enough to use buoyant weights for the in-water condition. If this is the case, use of the saturated unit weight would be more appropriate than the buoyant unit weight. Both of these variations are on the unconservative side in the sense that both indicate embedment is more effective than assumed herein. Neither variation is recommended for this admittedly approximate procedure.

Any lateral earth pressures against the riser walls parallel to the direction of motion, and any frictional stresses due to these lateral pressures are neglected as being too unreliable. That is, the maximum resultant pressure difference, p_{max}, is determined without benefit of resistance due to frictional forces.

With p_{max}, M_o'', and V_o'' known - the internal strength and external stability requirements of the effective embedded portion of the riser may be investigated. Note that the maximum vertical moment, M_x, in a partially embedded riser may occur either within the embedded portion or at the base of the embedded portion.

FOUNDATION INFLUENCE ADJUSTMENTS

General Comments

Two assumptions are fundamental to the procedures presented thus far. These are:

(1) the free-field ground motion at the site is identical to the bedrock motion at the site, and

(2) the motion experienced at the base of a riser is identical to the free-field ground motion.

Free-field ground motion is defined as the motion that would occur at the site at the elevation of the base of the riser if no structure was present.

Neither of these assumptions is usually fully satisfied. In a strict sense, satisfaction would require that the bedrock surface be located at the riser base elevation and that the riser be bonded to the rock and remain bonded during earthquake shaking. In the usual case, bedrock is at depth and the riser rests on a yielding foundation. The riser and foundation form a complex dynamic response system. The foundation, or site soil conditions, influence riser response. The riser in turn interacts with the foundation and modifies the ground motion at the site.

It is convenient to treat these two influences separately through the use of two additional factors which may be applied to equation 13 to obtain a better value for the total base shear, V_o. The two factors are:

(1) the soil profile factor, P, and

(2) the soil-structure interaction factor, I.

Hence the expression for total base shear, modified for foundation influence, becomes:

$$V_o = (ZSRCW_T)(PI) \tag{53}$$

The factors are evaluated below. The soil profile factor causes design values to be increased. The soil-structure interaction factor causes design values to be decreased. Their product, the foundation influence, is often a value nearly equal to unity. This suggests why both factors have sometimes been omitted.

Soil Profile Factor

The characteristics of the soil profile underlying the site influence the free-field ground motion. The soil profile factor, P, recognizes magnification of the free-field ground motion can occur when bedrock does not reach the riser base elevation. Three soil profile types are defined and P values are assigned. These definitions follow.

Soil profile type 1.--This type encompasses two profiles, they are:

(1) bedrock of any characteristic, either shale-like or crystalline, reaching riser base elevation, or

(2) stiff soil conditions where the soil depth is less than 200 feet and the soils overlying bedrock are stable deposits of sands, gravels, or stiff clays.

Soil profile type 2.--A profile with deep cohesionless or stiff clay conditions; includes sites where the soil depth exceeds 200 feet and the soils overlying bedrock are stable deposits of sands, gravels, or stiff clays.

Soil profile type 3.--A profile with soft-to-medium stiff clays and sands, characterized by 30 feet or more of soft-to-medium stiff clays with or without intervening layers of sand or other cohesionless soils.

Soil profile factor values.--In locations where the soil profile is not known in sufficient detail to determine the profile type or where the profile does not fit any of the defined types, soil profile type 2 should be assumed. The factor values are:

Soil profile type 1; P = 1.0
Soil profile type 2; P = 1.2
Soil profile type 3; P = 1.5.

Soil-Structure Interaction Factor

Soil-structure interaction is the subject of much ongoing research. Interaction is a function of the structural characteristics of the riser and the properties of the local underlying soil deposits. As compared with a rigidly supported structure, soil-structure interaction of a flexibly supported structure results in an increase in the fundamental period of vibration, usually an increase in effective damping of vibrations, and an increase in displacements. Soil-structure interaction reduces the design values of base shears, lateral forces, and overturning moments from the values applicable to a rigid-base condition.

A simplified model is used to evaluate soil-structure interaction. The model accounts, in an approximate way, for free-field ground motion intensity and riser support flexibility. When a riser is not bonded to the foundation, earth shaking can produce a rocking of the riser which may cause portions of the base to lift from the ground for short durations of time. Such behavior leads to reduced riser response. The soil-structure interaction factor, I, recognizes this reduction. It is evaluated in terms of the eccentricity, e, of the line of action of the reaction from the centerline of the base. Soil-structure interaction factor values follow.

Riser on rock, anchored to develop tension:

$$I = 1.00$$

Riser not on rock, or on rock but not anchored to develop tension, when $e \leq B/6$:

$$I = 1.00$$

when $e > B/6$:

$$I = 1.00 - 0.90\left(\frac{e - B/6}{B}\right) \qquad (54)$$

Embedded riser, to obtain V_o' and V_o'':

$$I = 1.00$$

Use of the soil-structure interaction factor, I, is optional. Use of the factor in the design of unanchored, free-standing risers may result in an iterative process since the value of I cannot be determined until e is known.

DESIGN CONSIDERATIONS

Loading Conditions

Earthquake loading is normally combined only with the dead and other constant loading acting on the structure. The effects of earthquakes should be investigated for motion parallel to both principal axes. A decision is required concerning whether the riser should be analyzed for the in-air condition, for a partially submerged condition, and/or for the fully submerged condition.

It appears unrealistic to assume all risers are fully submerged at the time an earthquake occurs. It is suggested that dry-dam risers be analyzed for the in-air condition; also, that risers be analyzed for the in-air condition when the normal water surface is such that the height, H_n, of figure 7 is some small value, say equal to, or less than, about 0.20 H_s. It is suggested that risers be analyzed for the partially submerged condition, using $H_h = H_n$, when H_n is greater than about 0.20 H_s, but not more than about 0.80 H_s. Two analyses are suggested when H_n is greater than about 0.80 H_s: the partially submerged condition using $H_h = H_w$ with riser empty, and the fully submerged condition.

Allowable Stresses and Safety Factors

Allowable stresses in steel and concrete may be taken at 4/3 their normal working values when checking the adequacy of a design or determining the amount of reinforcing steel required. Thus for 4000 psi concrete and 40 ksi steel, the allowable flexural compressive stress in the concrete, shear stress in the concrete, and steel tensile stress are:

$$f_c = \frac{4}{3}(1600) = 2133.\ \text{psi}$$

$$v_c = \frac{4}{3}(70) = 93.\ \text{psi}$$

$$f_s = \frac{4}{3}(20) = 26.7\ \text{ksi}$$

Allowable contact bearing pressures may also be taken at 4/3 their normal values. For example, if the normal maximum allowables are 2000 psf for saturated conditions and 4000 psf for moist conditions, the maximum allowables for earthquake investigations are 2670 psf and 5330 psf respectively.

Determination of riser footing dimensions for external stability of the structure, for load combinations other than those including earthquakes, is normally accomplished by requiring the line of action of the reaction to lie within the middle third of the base. That is, contact bearing is everywhere compressive. With such criteria it is unnecessary to specify a minimum safety factor against overturning since overturning is automatically avoided. Note that for free-standing risers, the overturning factor of safety by this criteria is $\geq$ 3.0.

Under earthquake loading, contact bearing will often not be everywhere compressive. Rather, the bearing pressure diagram will be triangular over a portion of the available bearing area. Under these conditions the structure should be tested against overturning. The overturning factor of safety should not be less than about 3/4 x 3.0 = 2.25, say 2.0. For free-standing risers, a factor of safety of not less than 2.0 corresponds to a requirement that the line of action of the reaction lie within the middle one-half of the base.

The factor of safety against sliding, for load combinations other than those including earthquakes, is usually set at about 1.5. When earthquakes are included, the sliding factor of safety should not be less than about 3/4 x 1.5 = 1.125, but preferably it should be something higher than this value.

Stress Analyses for Vertical Bending

Vertical steel requirements in sidewalls and endwalls should be investigated. Sometimes it will be found that the stresses are such that the concrete will not crack. In this event, it may be assumed that the usual steel for temperature and shrinkage is sufficient.

Often, the earthquake bending moment at a section will cause stresses sufficient to crack the concrete. The vertical steel required in the wall under investigation may be determined by the approximate relation:

$$A_s = \frac{M_x}{f_s(B - t)} \tag{55}$$

where:

$A_s \equiv$ total vertical steel area required in the wall under investigation at the section under consideration, sq. in.

M_x ≡ earthquake bending moment at level x, ft kips

f_s ≡ allowable steel stress, ksi

B ≡ width of the riser parallel to the direction of motion at the section under consideration, ft

t ≡ wall thickness of walls normal to direction of motion, at section under consideration, ft

Equation 55 is quick, approximate, and usually conservative. Note that it would always be conservative if the tensile steel could all be the same distance (B - t/2) from the compression face. The equation does not include the effect of the structure weight above the section under consideration, nor does it adjust stresses to account for differences between c.g. distances and extreme fiber distances. The analysis below includes these effects and may be used as an alternate to equation 55. The analysis is still approximate in that it assumes a location of the axis of zero stress and it approximates the steel stress at the c.g. of steel. It is conservative for underreinforced sections.

With reference to figure 9, temporarily assume stresses are allowable stresses, hence:

$$K = \frac{f_c}{f_c + \frac{f_s}{n}} \equiv \text{ratio locating assumed zero stress axis}$$

Then assuming the resultant compression, C, acts only on the wall under consideration:

$$C = \frac{1}{2} f_c \left(1 + \frac{KB - t}{KB}\right) tL \tag{56}$$

and assuming the tensile steel is uniformly distributed in the wall under consideration then:

$$T = \frac{1}{2} f_s \left(1 + \frac{(1 - K)B - t}{(1 - K)B}\right) A_s \tag{57}$$

Bending relations may be determined from figures 9 and 10. The force system acting at the center of gravity of the section is translated to the equivalent force system acting at the location of the resultant tensile force. Thus:

$$M_{sx} = M_x + W_x\left(\frac{B - t}{2}\right) \tag{58}$$

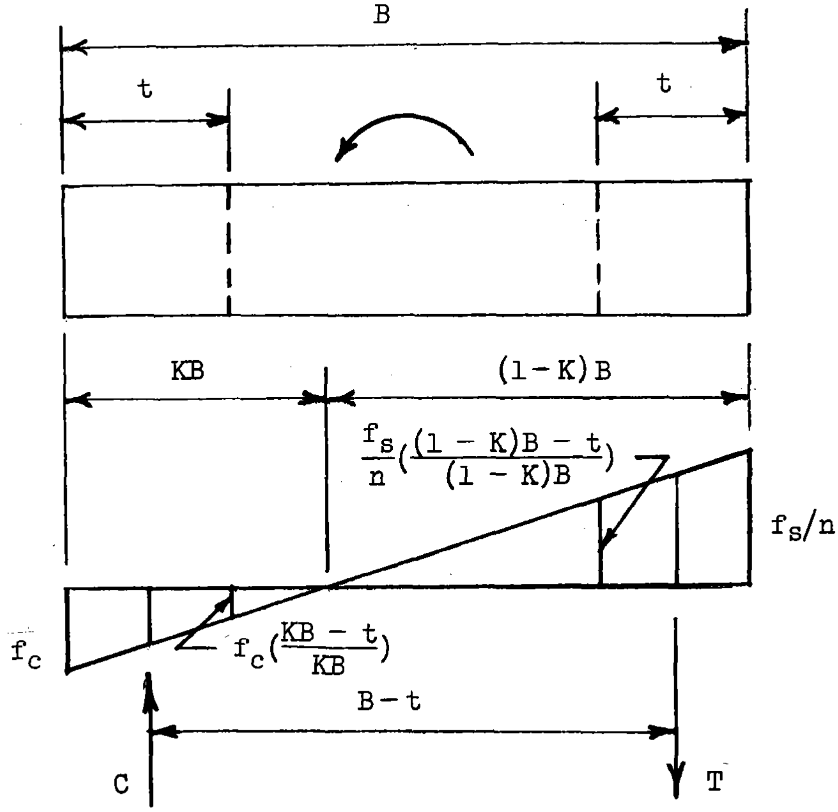


Figure 9. Assumed stress diagram.

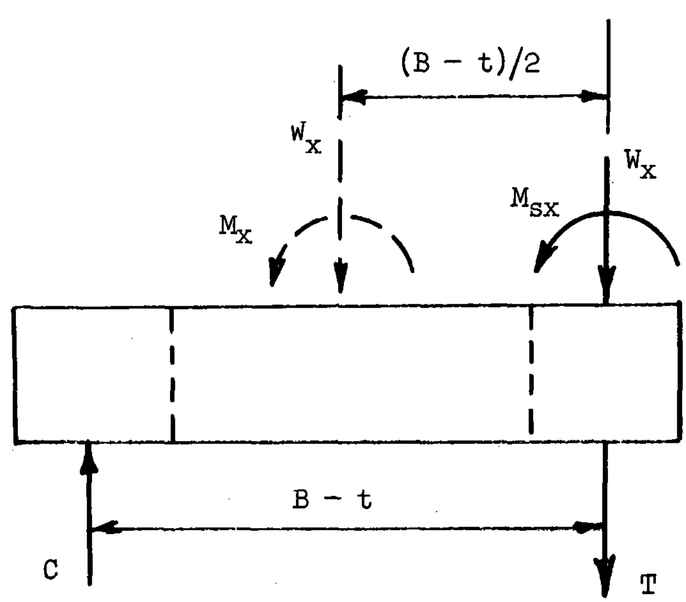


Figure 10. Bending at level x.

then the concrete stress may be determined from:

$$M_{sx} = C(B - t) = \frac{1}{2} f_c(1 + \frac{KB - t}{KB})tL(B - t) \qquad (59)$$

wherein units must be consistent. Note that the actual moment arm would be something in excess of (B - t). Similarly, either the tensile stress or the required tensile steel area may be determined from:

$$T = C - W_x \qquad (60)$$

or:

$$\frac{1}{2} f_s(1 + \frac{(1 - K)B - t}{(1 - K)B}) A_s = \frac{M_{sx}}{(B - t)} - W_x \qquad (61)$$

Note that the above analyses strictly apply only when both:

$$KB \geq t \quad \text{and} \quad (1 - K)B \geq t. \qquad (62)$$

Of the two relations, satisfaction of the second is the more important. If these relations are not sufficiently satisfied, then still more refined analyses may be used.

Stress Analyses for Horizontal Bending

Concrete stresses and horizontal steel requirements in sidewalls and endwalls should be investigated. Earthquake motion can induce lateral pressures that are significantly different in magnitude and/or sense from those treated in riser design when earthquake potential is not a consideration. Technical Release No. 30, "Structural Design of Standard Covered Risers," discusses the pipe-flow and no-flow loading conditions routinely treated. Various aspects of horizontal bending, in concert with earthquakes, are presented below.

Risers in air.--Figure 4 shows lateral forces and pressures increasing, and wall thicknesses decreasing, toward the top of the riser. Hence horizontal bending due to earthquake will be most severe in the top portions of the riser. Figure 11 portrays various considerations in horizontal bending. The lateral force, F_j, on any segment, j, is shown in (a) and again on the isolated segment in (b). The force, F_j, is carried in part by the forces, F_{js}, on the sidewalls and in part by the forces, F_{je}, on the endwalls as shown in (d). The sidewall force, F_{js}, is carried as a distributed unit pressure, w_{js}, on the sidewall as shown in (c) and (e). Sketch (f) indicates the closed-section bending loading. The closed-section reactions are assumed carried to ground by shear in the endwalls. Note that w_{js} is an inward load on one sidewall and an outward load on the other sidewall, also the senses reverse with time. From the figure, with consistent

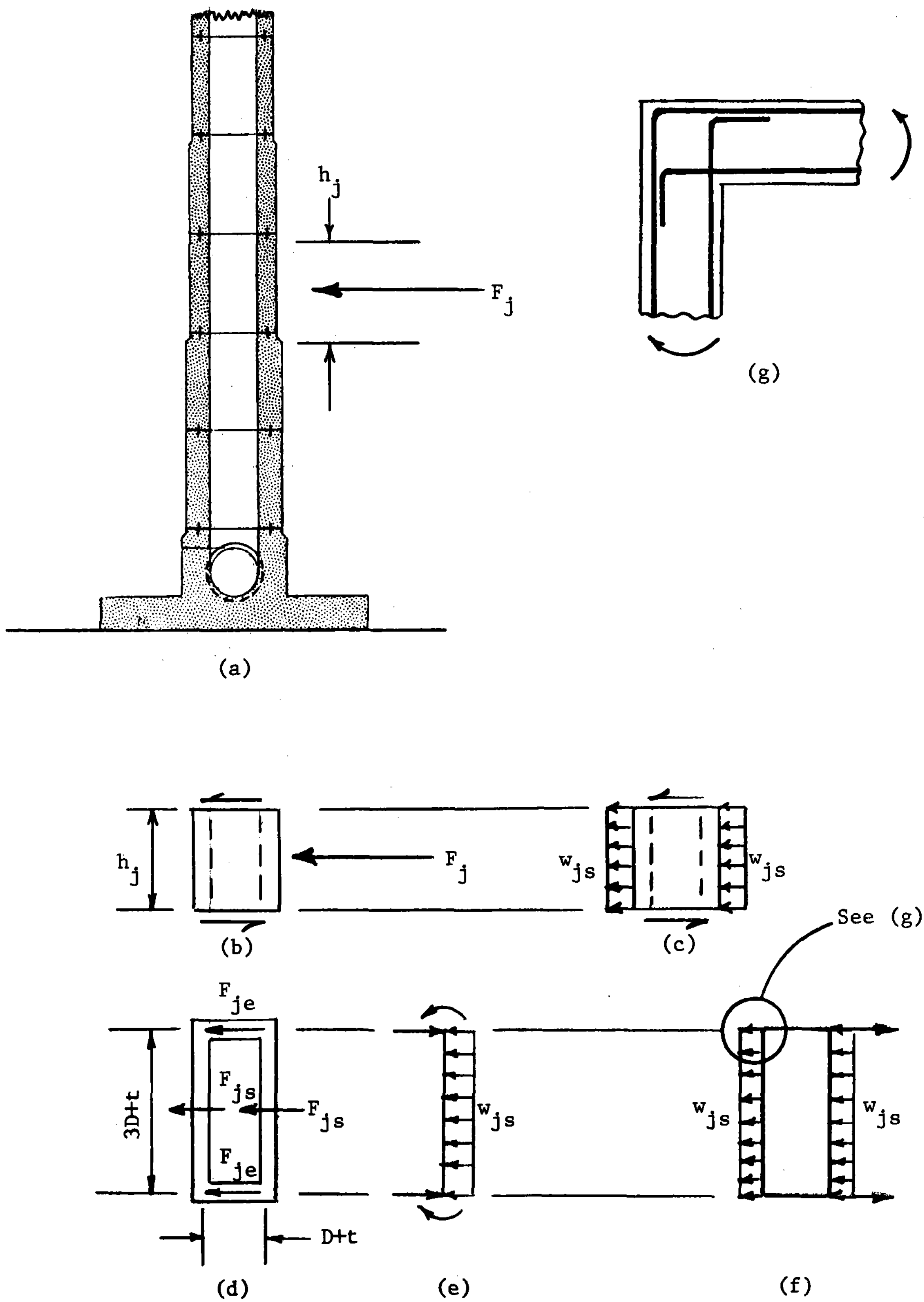


Figure 11. Horizontal closed-section bending.

units and assuming interior riser dimensions of D x 3D, the sidewall force is:

$$F_{js} = F_j\left(\frac{3D + t}{8D + 4t}\right) \tag{63}$$

and the unit pressure on the sidewall is:

$$w_{js} = \frac{F_{js}}{h_j(3D + t)} \tag{64}$$

Similar expressions may be written when the earthquake forces are normal to the endwalls.

With w_{js} determined, an indeterminate analysis of the indicated closed-section may be performed for a slice of unit thickness. The inward/outward loading produces horizontal bending moments. Some of these moments will be of opposite sense to those routinely considered. Some of the moments, although of the usual sense, may have magnitudes in excess of those routinely considered. Thus horizontal steel requirements, where earthquakes may be significant, deserve careful consideration. This includes not only area and spacing selection, but also location and development of the reinforcement. The steel layout for corner moments producing tension on the inside of the corner is particularly critical; sketch (g) shows a possible corner detail for this corner moment.

<u>Risers in water, partially submerged</u>.--The in-air portion of the riser is analyzed as discussed above. Horizontal bending investigations of the submerged portion of the riser must consider the hydrostatic pressures acting on the riser at the elevation under investigation. The combination of hydrostatic pressures and earthquake induced pressures may produce steel requirements in excess of those routinely determined.

<u>Risers in water, fully submerged</u>.--At any elevation under investigation, earthquake induced lateral pressures must be considered together with the lateral pressures routinely treated. Technical Release No. 30 discusses the difference between the pressures on the exterior and interior surfaces of the riser wall during pipe flow. These pressure differences produce net inward loadings. The magnitudes of these pressure differences depend on the flow velocity in the riser and hence sometimes are quite small. With earthquake motion, the sense of resultant moments at various sections may be either the same as, or the opposite of, those routinely determined. Again, steel selection and detailing should be meticulous.

Partially embedded risers, embedded portion.-- Figure 8 shows the assumed linearly varying resultant lateral earth pressures acting on the embedded portion of the riser. The resultant lateral earth pressure at any elevation is taken as the difference between simultaneous inward acting pressures on opposite sides of the riser. The mid-depth pressure on the side opposite the displacement side is taken as active pressure. The mid-depth pressure on the displacement side is taken as the sum of the resultant pressure, p_{max}, and the active pressure. The limiting value of the mid-depth pressure on the displacement side is passive pressure. The unit weight of the constructed fill is either its moist or its buoyant value as appropriate. As previously mentioned, more severe assumptions in evaluating the limiting value of p_{max} are possible. These may lead to greater resultant lateral earth pressures and greater horizontal bending.

For the partially submerged condition, hydrostatic pressures must be considered along with the earthquake lateral earth pressures. For the fully submerged condition, the net inward loading due to the pressure difference on exterior and interior wall surfaces during pipe flow must be considered along with the earthquake lateral earth pressures.

Shear Stress Analyses

Shearing stresses as a measure of diagonal tension will sometimes be critical in risers during earthquake shock. Two sources of stress should be considered. These are: shear stress due to closed-section bending, and shear stress due to shearing forces on horizontal planes.

Shearing stresses on horizontal planes may be assumed carried by the cross section webs, that is, the riser walls parallel to the direction of motion. The shear stresses are given approximately by:

$$v = \frac{V_x}{2t_w(B - \frac{t}{2})} \tag{65}$$

again, care must be exercised to maintain consistent units. Here

t_w ≡ wall thickness of walls parallel to direction of motion, ft

V_x ≡ earthquake shear force at level x, kips or lbs.

If the computed shear stress, v, exceeds the allowable shear stress, v_c, then web steel is required. The regular horizontal steel provided for horizontal bending stresses will often suffice. However, if the regular horizontal steel is utilized for shear, the inside steel will need to be extended to develop adequate end anchorages. If web steel is required, sufficient web steel should be provided to resist all of the shear without assistance from the concrete. This is true since diagonal tension stresses of sufficient magnitude to cause cracking may completely crack the concrete because of the alternating nature of the loading.

Thus, several functions of horizontal steel at a section are apparent:

(a) to resist horizontal bending stresses, both usual and earthquake induced,

(b) to carry shearing forces in the horizontal plane,

(c) to confine the concrete under reversal of loading, and

(d) to contain the vertical steel to prevent its buckling.

EXAMPLE COMPUTATIONS

Riser Data

The riser shown in figure 12 is analyzed for various seismic effects assuming ground motion is parallel to the endwalls. Assume the riser is in the reservoir of a class (b) structure located in western North Carolina. The foundation is soil profile type 1 with soil depth to bedrock less than 200 ft.

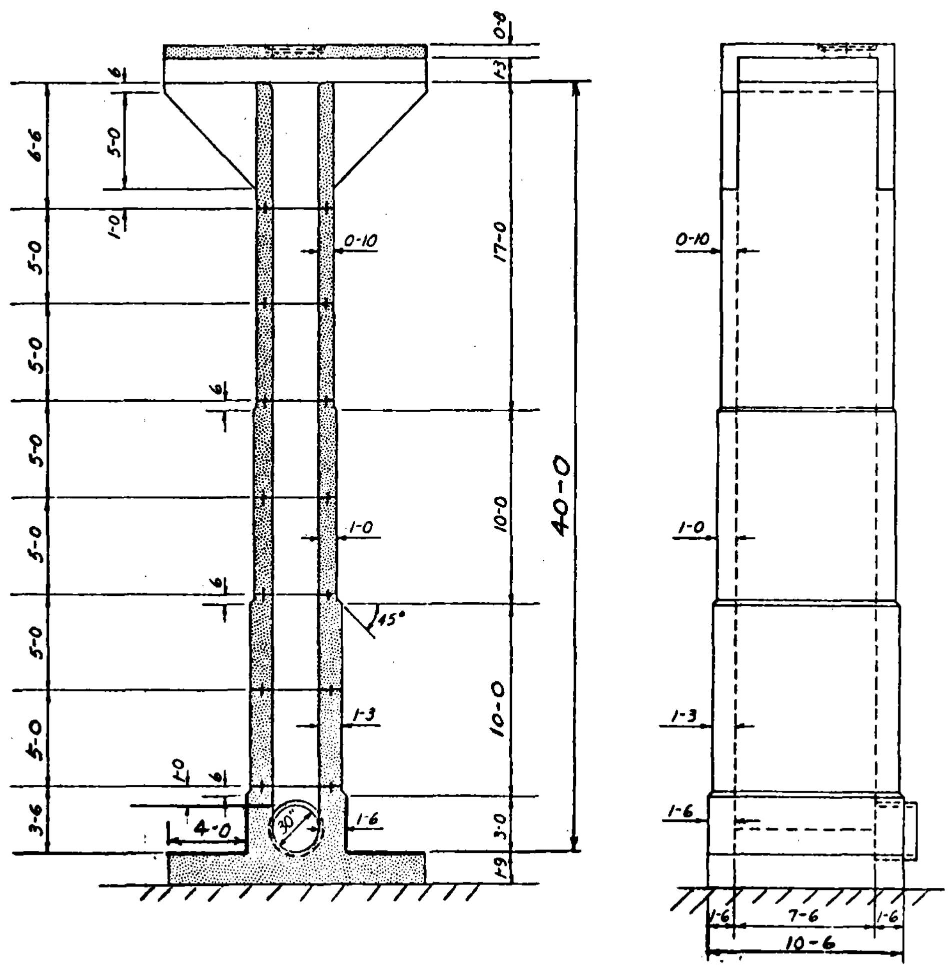


Figure 12. Riser for seismic analysis.

The riser is analyzed for three situations:

(a) free-standing, in-air condition,

(b) free-standing, in-water condition fully submerged, and

(c) partially embedded, in-water condition fully submerged.

In the analyses, the round bottom is neglected and local effects of the spigot wall fitting are neglected.

Analysis for Riser in Air

The soil profile is type 1, therefore the soil profile factor is P = 1.0. The soil-structure interaction factor is unknown. Will use I = 1.00 initially and make subsequent adjustments as may be indicated.

Computation of shears and moments.--Table 1 is used to develop various data and to tabulate resulting lateral forces, shears, and moments. Figure 13 defines the riser segments selected for the lumped-mass analysis.

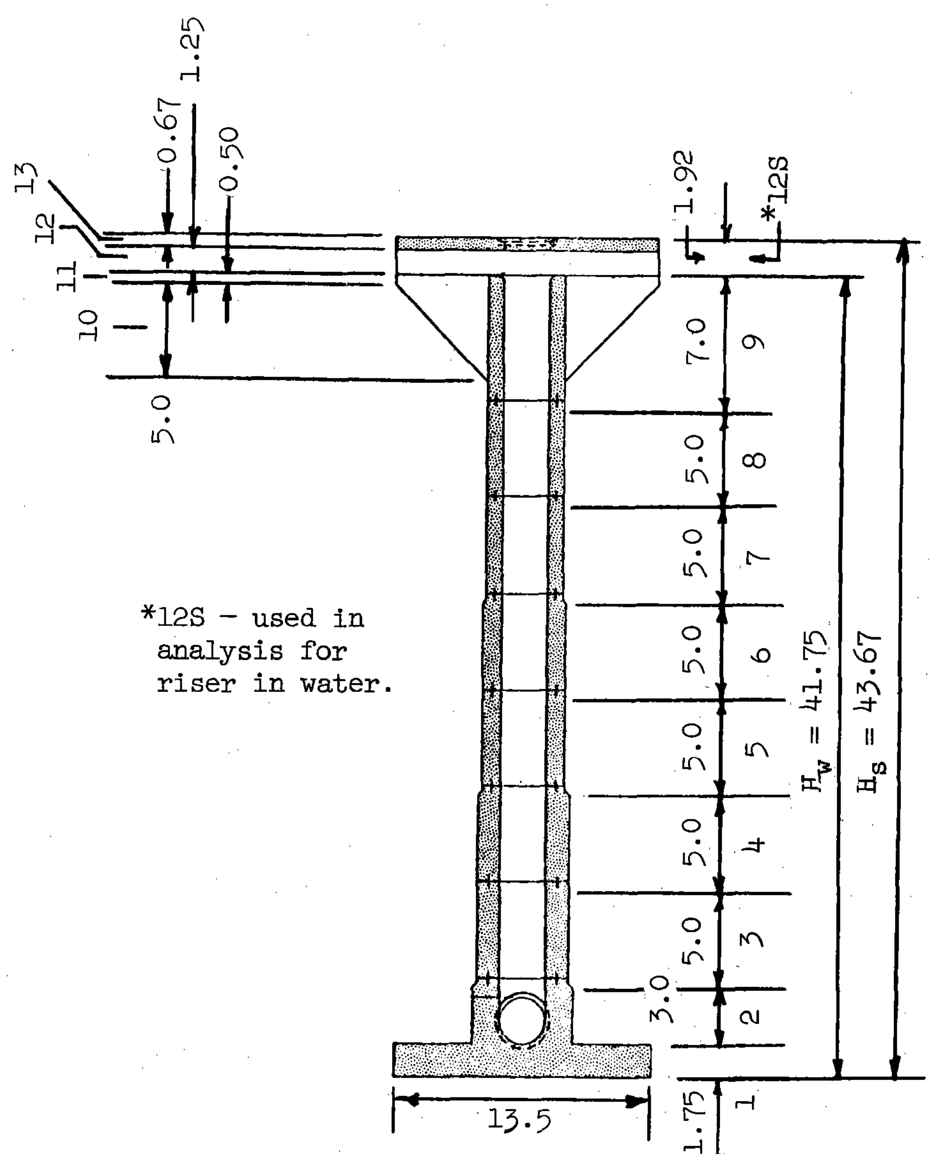


Figure 13. Riser segment selection.

Western North Carolina is in seismic zone 2, therefore Z = 0.50. The structure is a class (b) dam, therefore S = 1.50.

Using the modulus of elasticity of concrete as E = 525,000 ksf for 4000 psi concrete, g = 32.2 fps^2, and the summations of columns 6 and 10 of table 1, the fundamental period by equation 17, is:

$$T = \frac{2\pi}{3.567}\sqrt{\frac{3 \times 212.26}{525000 \times 32.2}} \times 256.75 = 0.17 \text{ sec.}$$

The effective width of the riser, B_e, parallel to the endwalls is determined using the summation of column 11 of table 1 as:

$$B_e = \frac{\Sigma B_i h_i}{H_w} = \frac{206.02}{41.75} = 4.93 \text{ ft}$$

By equation 18, the upper limit of the fundamental period of vibration, T, is in seconds:

$$T = 0.05 \times 43.67/\sqrt{4.93} = 0.98 \text{ secs.}$$

Note that the two values of T are quite different. This is not unexpected since equation 18 is more applicable to building frames than it is to tower structures. The value of T from equation 17 is used. Thus from equation 14, the base shear coefficient is:

$$C = 0.05/(0.17)^{1/3} = 0.090.$$

The total base shear is found from equation 53, thus:

$$\begin{aligned} V_o &= (0.50 \times 1.50 \times 2.0 \times 0.090 \times 212.26)(1.0 \times 1.0) \\ &= 0.135 \times 212.26 = 28.66 \text{ kips} \end{aligned}$$

The top lateral force is found from equation 21, thus:

$$F_t = 0.004\left(\frac{43.67}{4.93}\right)^2 28.66 = 0.31 \times 28.66$$

this is greater than 0.15 V_o so take:

$$F_t = 0.15 \times 28.66 = 4.30 \text{ kips}$$

The lateral force acting on any segment is found from equation 22, thus for segment 6, using column 12 of table 1:

$$F_6 = (28.66 - 4.30)\left(\frac{400.50}{3960.07}\right) = 2.46 \text{ kips}$$

The shear at any level, x, is found from equation 23. Column 15 of table 1 develops these shears. Note that the intent is to determine the shear at the bottom of each segment. The shears given at the bottoms of segments 10 and 11 do not quite fit this system because of the overlap between segments 9, 10, and 11.

Column 16 of table 1 gives the statical moment, M_s, at any level x. These statical moments could be determined by using the term within the braces of equation 27. However, since the statical moment at all levels x is desired, the basic procedure (modified for segments 9, 10, and 11) is to isolate a free body of the segment of interest and compute the statical moment at the bottom of the segment. This is computed as the statical moment at the top of the segment plus the shear at the top times the segment height plus the local lateral force times its moment arm about the bottom. The statical moments for segments 13, 12, 11, and 10 are determined below. These values are influenced by the segmentation selected near the top of the riser.

$$M_{s,13} = 4.30 \times 0.67 + 3.46 \times 0.67/2 = 4.0$$

$$M_{s,12} = 4.30 \times 1.92 + 3.46 \times 1.58 + 1.16 \times 1.25/2 = 14.4$$

$$M_{s,11} = 4.30 \times 2.42 + 3.46 \times 2.08 + 1.16 \times 1.125 + 0.32 \times 0.50/2$$
$$= 19.0$$

$$M_{s,10} = 4.30 \times 7.42 + 3.46 \times 7.08 + 1.16 \times 6.125 + 0.32 \times 5.25$$
$$+ 1.52 \times 10./3 = 70.3$$

The statical base moment reduction factor is computed from equation 26, thus:

$$J_o = 0.6/(0.17)^{1/3} = 1.08$$

which is greater than 1.0, so use:

$$J_o = 1.00$$

The statical moment reduction factor for moment at any level x is ordinarily computed by equation 28. However, from equation 28, since $J_o = 1.00$ then all:

$$J_x = 1.00$$

The moment at any level x is ordinarily computed by equation 27 or by:

$$M_x = J_x M_s \tag{66}$$

if all moments M_x are desired. In this case, since all $J_x = 1.00$, then:

$$M_x = M_s$$

Hence the values in column 16 of table 1 are repeated in column 18.

Table 1. Riser in air.

Segment i	h_i (ft)	B_i (ft)	L_i (ft)	w_i (kℓf)	W_i (kips)	H_i (ft)	H_w-H_i (ft)	I_i (ft^4)	$(H_w-H_i)^2\frac{h_i}{I_i}$ (ft^{-1})	$B_i h_i$ (ft^2)	$W_i H_i$ (kip-ft)	H_x (ft)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
Top Lateral Force	-	-	-	-	-	-	-	-	-	-	-	43.67
13	0.67	-	9.17	19.48	12.99	43.333	-	-	-	-	562.90	43.00
12	1.25	-	9.17	3.54	4.43	42.375	-	-	-	-	187.72	41.75
11	0.50	-	9.17	2.50	1.25	41.500	-	-	-	-	51.88	41.25
10	5.00	-	9.17	1.25	6.25	39.583	-	-	-	-	247.39	36.25
9	7.00	4.17	9.17	2.92	20.46	38.25	3.50	45.5	1.89	29.19	782.60	34.75
8	5.00	4.17	9.17	2.92	14.62	32.25	9.50	45.5	9.92	20.85	471.50	29.75
7	5.00	4.17	9.17	2.92	14.62	27.25	14.50	45.5	23.11	20.85	398.40	24.75
6	5.00	4.5	9.5	3.60	18.00	22.25	19.50	62.4	30.48	22.50	400.50	19.75
5	5.00	4.5	9.5	3.60	18.00	17.25	24.50	62.4	48.12	22.50	310.50	14.75
4	5.00	5.0	10.0	4.69	23.44	12.25	29.50	94.4	46.09	25.00	287.14	9.75
3	5.00	5.0	10.0	4.69	23.44	7.25	34.50	94.4	63.04	25.00	169.74	4.75
2	3.00	5.5	10.5	5.85	17.55	3.25	38.50	135.8	32.74	16.50	57.04	1.75
1	1.75	13.5	10.5	21.26	37.21	0.875	40.88	2153.	1.36	23.63	32.56	0.00
Σ					212.26				256.75	206.02	3960.07	

Segment i	I = 1.00					I = 0.90				
	F_j (kips)	V_x (kips)	M_s (ft-kips)	J_x -	M_x (ft-kips)	F_j (kips)	V_x (kips)	M_s (ft-kips)	J_x -	M_x (ft-kips)
	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)
Top Lateral Force	4.30	4.30	0.0	1.0	0.0	3.87	3.87	0.0	1.0	0.0
13	3.46	7.76	4.0	↓	4.0	3.11	6.98	3.6	↓	3.6
12	1.16	8.92	14.4	↓	14.4	1.04	8.02	13.0	↓	13.0
11	0.32	*9.24	*19.0	↓	*19.0	0.29	*8.31	*17.1	↓	*17.1
10	1.52	*10.76	*70.3	↓	*70.3	1.37	*9.68	*63.3	↓	*63.3
9	4.81	15.57	103.	↓	103.	4.33	14.01	92.7	↓	92.7
8	2.90	18.47	188.	↓	188.	2.61	16.62	169.	↓	169.
7	2.45	20.92	287.	↓	287.	2.20	18.82	258.	↓	258.
6	2.46	23.38	398.	↓	398.	2.21	21.03	358.	↓	358.
5	1.91	25.29	519.	↓	519.	1.72	22.75	467.	↓	467.
4	1.77	27.06	650.	↓	650.	1.59	24.34	585.	↓	585.
3	1.05	28.36	788.	↓	788.	0.95	25.20	709.	↓	709.
2	0.35	28.46	873.	↓	873.	0.32	25.61	786.	↓	786.
1	0.20	28.66	923.	1.0	923.	0.18	25.79	831.	1.0	831.
Σ	28.66					25.79				

*These values influenced by segment selection near top; values above and below are valid.

Before beginning various stress computations, investigate the value and effect of the soil-structure interaction factor. The overturning moment about the base is:

$$M_o = 923.\ \text{ft kips}$$

also:

$$W_T = 212.26\ \text{kips}$$

$$B = 13.5\ \text{ft}$$

The eccentricity of the total weight, W_T, from the footing centerline is:

$$e = \frac{923.}{212.26} = 4.35\ \text{ft}$$

and:

$$B/6 = 13.5/6. = 2.25\ \text{ft}$$

Therefore from equation 54, the interaction factor is:

$$I = 1.00 - 0.90\left(\frac{4.35 - 2.25}{13.5}\right) = 0.86$$

Use of I = 0.86 in equation 53 will reduce the previously determined lateral forces, shears, and moments proportionately. This will produce a new reduced eccentricity of:

$$e = \frac{923.\ \text{x}\ 0.86}{212.26} = 3.74\ \text{ft}$$

which in turn gives a new increased interaction factor of:

$$I = 1.00 - 0.90\left(\frac{3.74 - 2.25}{13.5}\right) = 0.90$$

Another cycle would produce a slightly smaller value of I, etc. The value I = 0.90 is used in the remaining investigation. This produces results that are slightly on the conservative side. Columns 19, 20, 21, and 23 give the reduced results.

Stress computations.--Various vertical bending stress conditions are investigated in the riser at the top of the footing.

Determine if moment, M_x, is sufficient to cause cracking of a plain concrete section. Test by using:

$$f_{min} \simeq \frac{W_x}{A_g} - \frac{M_x \frac{B}{2}}{I_g} \qquad (67)$$

where

f_{min} ≡ minimum extreme fiber stress

A_g ≡ gross area of concrete, ft^2

I_g ≡ gross moment of inertia of section neglecting steel, ft^4

Here:

$$W_x = 212.26 - 37.21 = 175 \text{ kips}$$

$$M_x = 786. \text{ ft kips}$$

$$V_x = 25.61 \text{ kips}$$

$$A_g = 10.5 \times 5.5 - 7.5 \times 2.5 = 39.0 \text{ ft}^2$$

$$I_g = \frac{1}{12}(10.5 \times \overline{5.5}^3 - 7.5 \times \overline{2.5}^3) = 135.8 \text{ ft}^4$$

$$B = 5.5 \text{ ft}$$

$$L = 10.5 \text{ ft}$$

$$t = 1.5 \text{ ft}$$

$$f_{min} = \frac{175.}{39.} - \frac{786. \times 5.5/2.}{135.8} = -11.43 \text{ ksf} = -79. \text{ psi}$$

This is relatively low tensile stress. Assuming $f'_c = 4000$ psi, the tensile strength is about 400 psi. Therefore the section probably will not crack due to this loading.

Assuming the concrete does crack, determine bending adequacy and/or requirements. Take $f'_c = 4000$ psi and $f_y = 40$ ksi.

By equation 55, the required steel area is:

$$A_s = \frac{786.}{26.7(5.5 - 1.5)} = 7.36 \text{ sq inches in the sidewall.}$$

Compare with results using equations 56 through 61. The equivalent moment about the tension steel is, by equation 58:

$$M_{sx} = 786. + 175.(5.5 - 1.5)/2$$

$$= 786. + 350. = 1136. \text{ ft kips}$$

Noting:

$$K = \frac{2133}{2133 + \frac{26700}{8}} = 0.39$$

then by equation 59, the concrete stress is obtained as:

$$1136. = \frac{1}{2} f_c(1 + \frac{.39 \times 5.5 - 1.5}{.39 \times 5.5})1.5 \times 10.5(5.5 - 1.5)$$

or:

$$= 0.65 f_c \times 15.75 \times 4. = 40.95 f_c$$

so:

$$f_c = 27.7 \text{ ksf} = 0.193 \text{ ksi} = 193. \text{ psi.}$$

Since this is a relatively small stress, concrete stresses are not investigated further. However, if desired, concrete stresses could be computed more accurately by adjusting the location of the zero stress axis and performing another cycle of analysis.

Now by equation 61:

$$\frac{1}{2} f_s (1 + \frac{.61 \times 5.5 - 1.5}{.61 \times 5.5}) A_s = \frac{1136.}{5.5 - 1.5} - 175.$$

or

$$0.78 \; f_s \; A_s = 284. - 175. = 109. \text{ kips}$$

the required steel area thus is:

$$A_s = \frac{109.}{0.78 \times 26.7} = 5.23 \text{ sq inches in the sidewall.}$$

This compares with the more approximate value previously found as 7.36 sq inches.

Note that temperature and shrinkage would require at least:

$$2 \times 0.002 \times 1.5 \times 10.5 \times 144. = 9.07 \text{ sq inches}$$

of vertical steel in the sidewall for this riser. Of course, because of the wall-to-footing connection, vertical bending requirements would probably cause still more steel to be present.

Horizontal bending stresses due to earthquake motion are maximum at the top of the riser walls. The sidewalls in the top segment of the riser, segment 9, each carry the force, F_{js}. By equation 63, this is:

$$F_{js} = 4.33(\frac{7.5 + 10/12}{20.0 + 40/12}) = 1.55 \text{ kips}$$

and by equation 64, the distributed unit pressure on the sidewalls is:

$$w_{js} = \frac{1.55}{7.0(7.5 + 10/12)} = .027 \text{ ksf} = 27. \text{ psf}$$

As an outward loading on one sidewall and an inward loading on the other, this pressure will produce corner moments with tension on the inside of the corner and mid-span moments with tension on the outside of the sidewall. The magnitudes of the moments can be found from an indeterminate analysis. An indeterminate analysis is not performed herein. Neither the corner moment nor the sidewall mid-span moment will exceed the simple span moment of:

$$M = \frac{w\ell^2}{8} = \frac{27(7.5 + 10/12)^2}{8} = 234 \text{ ft lbs.}$$

The maximum concrete tensile bending stress is thus less than:

$$f_t = \frac{6M}{bt^2} = \frac{6(234 \times 12)}{12 \times \overline{10}^2} = 14 \text{ psi}$$

Hence horizontal bending is not a problem for this riser.

Shear stress on the horizontal plane at the top of the footing, is:

$$v = \frac{25.61}{2 \times 1.5(5.5 - 1.5/2)} = 1.80 \text{ ksf} = 12 \text{ psi}$$

which is another small value.

Bearing computations.--The overturning moment about the base is

$$M_o = 831. \text{ ft kips}$$

also

$$W_T = 212.26 \text{ kips}$$

$$B = 13.5 \text{ ft}$$

$$L = 10.5 \text{ ft}$$

The eccentricity of the total weight, W_T, from the footing centerline is

$$e = \frac{831.}{212.26} = 3.92 \text{ ft}$$

This is greater than:

$$\frac{B}{6} = \frac{13.5}{6.} = 2.25 \text{ ft}$$

so that the bearing pressure diagram is triangular. The maximum bearing pressure, p, is given by:

$$\frac{1}{2}\, p \times 3(\frac{B}{2} - e)\, L = W_T \tag{68}$$

or

$$p = \frac{2 \times 212.26}{3(13.5/2 - 3.92)10.5} = 4.76 \text{ ksf} = 4760 \text{ psf}$$

Thus, if the allowable bearing pressure for moist conditions is:

$$\frac{4}{3} \times 4000 = 5330 \text{ psf}$$

The foundation is adequate for bearing.

The factor of safety against overturning is:

$$SF_o = \frac{212.26 \times 13.5/2.}{831.} = 1.72$$

which is less than the suggested 2.0.

Sliding.--Assume the coefficient of friction, soil to concrete, is 0.35. Thus, the factor of safety against sliding is

$$SF_s = \frac{(212.26)(0.35)}{25.79} = 2.88$$

which is quite adequate.

Conclusions.--It is recognized that the riser should be analyzed for effects of earthquake motion parallel to the sidewalls before final conclusions are drawn. Also, additional analysis should be considered for stress conditions at various other locations in the riser. Tentative conclusions, based on the analyses already made, would indicate that for this riser in a real design situation, the riser walls are probably adequate. Both vertical and horizontal bending stresses are low. Shear stress is low. Wall steel selected in routine design would probably suffice. Bearing pressures are adequate. Sliding safety is quite acceptable. The factor of safety against overturning is less than desirable. Increasing the riser footing projections would be a consideration.

Briefly, with respect to earthquake motion parallel to the sidewalls, note the following. The factor of safety against overturning will be even less satisfactory than above since the overturning moment is probably larger while the footing dimension parallel to motion is less. Again, an increase in the size of the riser base is indicated if the riser is to be satisfactory as a free-standing tower structure in the given environment.

Analysis for Riser in Water, Fully Submerged.

Again, P = 1.0 and will use I = 1.0 initially.

Computation of shears and moments.--Table 2 is used to develop data for the determination of the distributed added mass or weight to be combined with the weight of the riser plus water inside the riser. The table is also used to tabulate resulting lateral forces, shears, and moments.

It is convenient to treat the added weight above the crest of the riser separately from the weight of the riser cover slab and cover slab walls. Therefore, a special segment 12S is introduced in Table 2 and shown in figure 13.

Column 2 of table 2 repeats column 3 of table 1 in order to compute the β_i values of column 3. β_i is computed from equation 44. The base values ζ_i, of the added mass ellipses are in turn computed from equation 42 and are given in column 4. The ellipse ordinate values, r, are computed from equation 35 and are given in column 6. Note that for a fully submerged riser, $H_h = H_s$. The ellipse abscissa values, z_i, are next computed from equation 41 and are tabulated in column 7.

The added weight per ft of height for each segment is computed from equation 40. The total added weight for each segment is found from equation 45. The results are given in columns 9 and 11. The weight of water inside the riser is determined for each segment and given in column 13. The weight of water inside the riser for segment 12S is computed using the tabulated h_i of 1.92 ft. This introduces slight error on the conservative side but is used for convenience. The total segment weight, W_i, is given in column 15, it is the sum of columns 11, 13, and 14. Note that in-air weights are used in column 14 not buoyant weights. Riser response is a function of mass. The in-air weight is an index of that mass.

The value of the period of vibration of the riser-water system is required. From equation 48:

$$\beta_e = \frac{4.93}{43.67} = 0.113$$

Hence from equation 46, the fundamental period of vibration of the fully submerged riser is:

$$T_{wf} = (1.46 - 0.77 \times 0.113 + 0.70 \times \overline{0.113}^2) \times 0.17$$

$$= 1.38 \times 0.17 = 0.23 \text{ sec.}$$

From this point on, the procedure is the same as that for the riser in air. From equation 14, the base shear coefficient is:

$$C = 0.05/(0.23)^{1/3} = 0.082$$

The total base shear is, from equation 53:

$$V_o = (0.50 \times 1.50 \times 2.0 \times 0.082 \times 344.05)(1.0 \times 1.0)$$

$$= 0.123 \times 344.05 = 42.32 \text{ kips}$$

The top lateral force is, from equation 21:

$$F_t = 0.004\left(\frac{43.67}{4.93}\right)^2 42.32 = 0.31 \times 42.32$$

since this is more than $0.15V_o$, use:

$$F_t = 0.15 \times 42.32 = 6.35 \text{ kips}$$

The lateral force acting on any segment is found from equation 22. The values are given in column 17 of table 2. For example for segment 6:

$$F_6 = (42.32 - 6.35)\frac{729.80}{6381.56} = 4.11 \text{ kips}$$

The shear at any level x is found from equation 23. Column 19 of Table 2 develops these shears. Note that the computed shears in the region of the cover slab walls are influenced by the manner of selecting segments. Column 20 gives the statical moment, M_s, at any level x. These statical moments can be determined by using the term within the braces of equation 27, or they can be computed directly from statics by isolating repetitive free body diagrams.

The statical base moment reduction factor is computed from equation 26, thus

$$J_o = 0.6/(0.23)^{1/3} = 0.98$$

With J_o less than 1.0, values of J_x could be computed from equation 28 and values of M_x could be computed from either equation 27 or equation 65. However, since 0.98 is very close to unity, for convenience moments are not reduced from their statical values. This introduces errors in M_x values on the conservative side ranging from 0 at the top to 2 percent at the bottom. Thus say all $J_x = 1.00$ and use:

$$M_x = M_s$$

Hence the values in column 20 are repeated in column 22 of Table 2.

Before beginning various stress computations, investigate the value and effect of the soil-structure interaction factor. The overturning moment about the base is:

$$M_o = 1316. \text{ ft kips}$$

Uplift pressures will exist under the riser footing. Hence the submerged or buoyany weight of the riser should be used to obtain contact bearing pressures. The buoyant weight, W_B, is:

$$W_B = W_{T_a}\left(\frac{150 - 62.4}{150}\right) \qquad (69)$$

Table 2. Riser in water, fully submerged.

Segment i	B_i (ft)	β_i -	ζ_i -	H_i (ft)	r -	ε_i -	L_i (ft)	w_{ai} (klf)	h (ft)	W_{ai} (kips)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
Top Lateral Force	-	-	-	-	-	-	-	-	-	-
13	-	-	-	43.333	-	-	-	-	-	-
12s	4.17	0.096	0.075	42.72	0.978	0.016	9.17	0.40	1.92	0.77
12	-	-	-	42.375	-	-	-	-	-	-
11	-	-	-	41.50	-	-	-	-	-	-
10	-	-	-	39.583	-	-	-	-	-	-
9	4.17	0.096	0.075	38.25	0.876	0.036	9.17	0.90	7.00	6.30
8	4.17	0.096	0.075	32.25	0.738	0.051	9.17	1.27	5.00	6.35
7	4.17	0.096	0.075	27.25	0.624	0.059	9.17	1.47	5.00	7.35
6	4.5	0.103	0.080	22.25	0.510	0.069	9.5	1.79	5.00	8.95
5	4.5	0.103	0.080	17.25	0.395	0.073	9.5	1.89	5.00	9.45
4	5.0	0.115	0.089	12.25	0.281	0.085	10.0	2.32	5.00	11.60
3	5.0	0.115	0.089	7.25	0.166	0.088	10.0	2.40	5.00	12.00
2	5.5	0.126	0.098	3.25	0.074	0.098	10.5	2.80	3.00	8.40
1	13.5	0.309	0.228	0.875	0.020	0.228	10.5	6.52	1.75	11.41

Segment i	w_{wi} (klf)	W_{wi} (kips)	W_i(air) (kips)	W_i kips	W_iH_i (kip-ft)	I = 1.00 F_j (kips)	H_x (ft)	V_x (kips)	M_s (ft-kips)	J_x -	M_x (ft-kips)
	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)
Top Lateral Force	-	-	-	-	-	6.35	43.67	6.35	0.0	1.0	0.0
13	-	-	12.99	12.99	562.90	3.17	43.00	9.52	5.3	↓	5.3
12s	1.17	2.25	-	3.02	129.01	0.73	41.75	-	-	↓	-
12	-	-	4.43	4.43	187.72	1.06	41.75	11.31	18.16	↓	18.6
11	-	-	1.25	1.25	51.88	0.29	41.25	*11.60	*24.3	↓	*24.3
10	-	-	6.25	6.25	247.39	1.39	36.25	*12.99	*86.9	↓	*86.9
9	1.17	8.19	20.46	34.95	1336.84	7.54	34.75	20.53	133.	↓	133.
8	↓	5.85	14.62	26.82	864.95	4.88	29.75	25.41	248.	↓	248.
7	↓	↓	14.62	27.82	758.10	4.27	24.75	29.68	386.	↓	386.
6	↓	↓	18.00	32.80	729.80	4.11	19.75	33.79	544.	↓	544.
5	↓	↓	18.00	33.30	574.43	3.24	14.75	37.03	721.	↓	721.
4	↓	↓	23.44	40.89	500.90	2.82	9.75	39.85	914.	↓	914.
3	↓	5.85	23.44	41.29	299.35	1.69	4.75	41.54	1117.	↓	1117.
2	1.17	3.51	17.55	29.46	95.75	0.54	1.75	42.08	1243.	↓	1243.
1	-	-	37.21	48.62	42.54	0.24	0.00	42.32	1316.	1.0	1316.
Σ		-	212.26	344.05	6381.56	42.32					

*These values are influenced by segment selection near top of riser; values above and below are valid.

where:

$$W_{T_a} \equiv \text{the weight of the riser in air}$$

thus:

$$W_B = 212.26\left(\frac{87.6}{150}\right) = 123.96 \text{ kips}$$

The eccentricity of the effective weight from the centerline of the footing is:

$$e = \frac{1316.}{123.96} = 10.62 \text{ ft}$$

this is greater than:

$$\frac{B}{2} = \frac{13.5}{2} = 6.75 \text{ ft}$$

Thus the riser is unstable for this overturning moment.

At incipient overturning $e = B/2$, the soil-structure interaction factor is, from equation 54:

$$I = 1.00 - 0.90\left(\frac{6.75 - 2.25}{13.5}\right) = 0.70$$

As a "worst case" test, take $e = B/2$ and $I = 0.70$. This will produce a new reduced eccentricity of:

$$e = \frac{1316. \times 0.70}{123.96} = 7.43 \text{ ft}$$

which is still greater than B/2. Thus this riser will be unstable against overturning if subjected to the assumed earthquake shock when the riser is fully submerged.

For purposes of stress computations and comparisons, the value $I = 1.00$ is used in the remaining investigation. Figure 14 shows plots of shears and moments for the riser in-air and for the riser in-water, fully submerged. The curves are shown for $I = 1.00$ for both conditions. The ratio of overturning moments, M_o, in the two cases is:

$$\frac{1316.}{923.} = 1.43$$

and the ratio of base shears, V_o, is:

$$\frac{42.32}{28.66} = 1.48.$$

Stress computations.--Again various vertical bending stress conditions in the riser at the top of the footing are investigated.

By equation 55, the required steel area is:

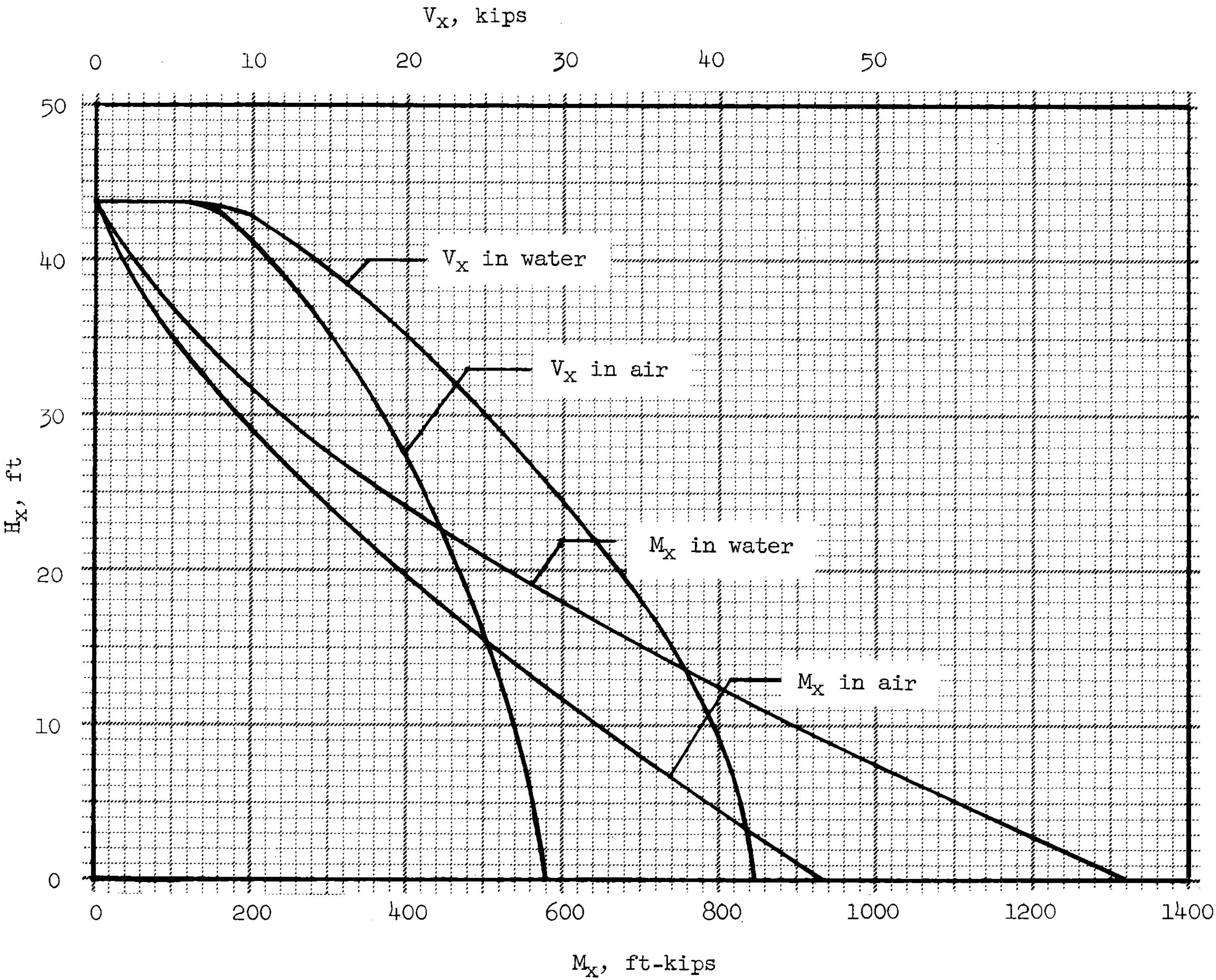


Figure 14. Shear and moment curves for riser in air and riser fully submerged, I = 1.00.

$$A_s = \frac{1243.}{26.7(5.5 - 1.5)} = 11.64 \text{ sq inches in the sidewall.}$$

By equation 58, the equivalent moment about the tension steel is

$$M_{sx} = 1243. + 102.2(5.5 - 1.5)/2$$
$$= 1243. + 204. = 1447. \text{ ft kips}$$

The buoyant weight of the riser above the section is used. This is the worst condition as regards steel stresses. The buoyant weight is:

$$W_b = 175.05\left(\frac{150 - 62.4}{150}\right) = 102.2 \text{ kips}$$

Note that the weight of the water inside the riser is not included, neither is the weight of the added mass included in the computation.

By equation 59, the concrete stress is obtained as

$$1447. = \frac{1}{2} f_c\left(1 + \frac{.39 \times 5.5 - 1.5}{.39 \times 5.5}\right)1.5 \times 10.5(5.5 - 1.5)$$

or:

$$1447. = 0.65\ f_c \times 15.75 \times 4. = 40.95\ f_c$$

so:

$$f_c = 35.3 \text{ ksf} = 0.245 \text{ ksi} = 245. \text{ psi}$$

By equation 61, vertical steel requirements are obtained as

$$\frac{1}{2} f_s\left(1 + \frac{.61 \times 5.5 - 1.5}{.61 \times 5.5}\right) A_s = \frac{1447.}{5.5 - 1.5} - 102.2$$

or:

$$0.78\ f_s\ A_s = 361.75 - 102.2 = 259.55 \text{ kips}$$

so:

$$A_s = \frac{259.55}{0.78 \times 26.7} = 12.46 \text{ sq inches}$$

Thus equation 57 can indicate a greater steel requirement than equation 51. This is due to the stress adjustment accounting for differences between c.g. distances and extreme fiber distances. If buoyant weight had not been used:

$$M_{sx} = 1243. + 175.(5.5 - 1.5)/2 = 1243. + 350. = 1593. \text{ ft kips}$$
$$f_c = 1593./40.95 = 38.9 \text{ ksf} = .270 \text{ ksi} = 270 \text{ psi}$$
$$0.78\ f_s A_s = \frac{1593.}{5.5 - 1.5} - 175. = 223.2 \text{ kips}$$
$$A_s = 223.2/(0.78 \times 26.7) = 10.72 \text{ sq inches}$$

Considering horizontal bending, the earthquake induced lateral pressures on the top segment of the riser, segment 9, are determined as follows. By equation 63:

$$F_{js} = 7.54\left(\frac{7.5 + 10/12}{20.0 + 40/12}\right) = 2.69 \text{ kips}$$

by equation 64:

$$w_{js} = \frac{2.69}{7.0(7.5 + 10/12)} = .046 \text{ ksf} = 46 \text{ psf}$$

By Technical Release No. 30, page 1-3, the pressure difference on the riser walls during pipe flow is a function of the velocity head in the riser and the distance the section under investigation is below the crest of the riser inlet.

To accurately determine stress conditions due to horizontal bending, the velocity in the riser is needed and indeterminate analyses combining the earthquake and pipe flow loadings would be required. In lieu of such analyses, since the velocity is unknown, the following bounds are noted. Taking the velocity very low, say zero, the 46 psf outward loading on a sidewall would cause a maximum concrete tensile bending stress that is less than (see computations for the riser in air):

$$f_t = 14(46/27) = 24 \text{ psi}$$

which is too small to be of concern.

Taking the velocity in the riser at its maximum value for standard risers, the maximum pressure difference, from Technical Release No. 30, is 6 x 0.96 x 62.4 = 359 psf. Combining these pressure differences with the earthquake induced pressures, the inward loadings on the riser walls are thus: 359 + 46 = 405 psf on one sidewall, 359 - 46 = 313 psf on the other sidewall, and 359 psf on the endwalls. As a standard riser, the top 6 feet of the riser walls would have been designed for a constant inward loading all around the walls of at least 6 x 62.4 = 374 psf. Steel requirements in the sidewalls, at mid-span and at the corners, due to the varying inward loadings will slightly exceed the requirements determined routinely for the standard riser loading. Probably, because of temperature and shrinkage, the steel routinely selected for this region of the riser walls has capacity in excess of need. For lack of analysis, the sidewall steel could be increased in the ratio of 405/374 = 1.08.

Shear stress on the horizontal plane at the top of the footing, neglecting the round bottom is:

$$v = \frac{42.32 - 0.24}{2 \times 1.5(5.5 - 1.5/2)} = 2.95 \text{ ksf} = 0.021 \text{ ksi} = 21 \text{ psi}$$

Bearing computations.--As previously determined, the riser is unstable against overturning. The eccentricity of the effective weight from the centerline of the footing computes as e = 10.62 ft which is greater than B/2 = 6.75 ft.

Note that the computed factor of safety against overturning is:

$$SF_o = \frac{123.96 \times 6.75}{1316} = 0.64$$

Sliding.--The computed factor of safety against sliding is

$$SF_s = \frac{123.96 \times 0.35}{42.32} = 1.03$$

which is less than the suggested 1.125.

Conclusions.--Based on the analyses thus far, the riser design is unacceptable. Although the design for internal strength is adequate, or can easily be made so by providing sufficient vertical and/or horizontal steel, external stability is lacking.

To make this riser externally stable on a yielding foundation, the footing would need to be increased considerably. Alternately, if the foundation were sufficiently competent rock, which is not the case for this example, the riser would need to be adequately joined to the rock.

A further alternative would be to sufficiently embed the riser as described in the earlier section.

One additional observation is perhaps worthwhile. The riser, if located in the reservoir area, would have been designed for moment due to wind. The wind moment would be, using a weighted sidewall width of about 9.50 ft:

$$M_w = \frac{0.050 \times 9.50}{2} \times \overline{43.67}^2 = 453. \text{ ft kips}$$

This is about one-half of the computed earthquake overturning moment for the riser in air. It is about one-third of the computed earthquake overturning moment for the riser in water.

Analysis for Partially Embedded Riser

The partially embedded riser is analyzed for the in-water condition, fully submerged. The riser is embedded to 20.0 ft above the floor (invert) of the riser, see figure 7. Additional required data is:

$$H_e = 20.0 + 1.75 = 21.75 \text{ ft}$$

$$\gamma_b = 67.6 \text{ pcf}$$

$$K_p = 2.5 \quad \text{and} \quad K_a = 0.40$$

From equation 50 with $H_t = 43.67$ ft, then

$$H_s = 43.67 - \frac{2}{3}(21.75) = 29.17 \text{ ft}$$

Recognizing the arbitrary nature of the above computation, the value will be adjusted for convenience of computation to

$$H_s = 28.92 \text{ ft}$$

and therefore, from figure 8:

$$H_r = 43.67 - 28.92 = 14.75 \text{ ft}$$

These values allow easy use of the previously selected segments.

Analysis of projecting portion of riser.--The analysis of the riser above the effective embedment depth, H_r, proceeds as if the riser had an actual height of H_s. The soil profile factor for the site is P = 1.0. The soil-structure interaction factor is I = 1.0 for embedded riser analyses. Table 3 is used to develop various data including the determination of the weight of the riser, the weight of added mass due to the surrounding water, and the weight of the water inside the riser. The table columns are ordered for a direct solution of the in-water condition. The table is also used to tabulate resulting lateral forces, shears, and moments.

The fundamental period of vibration in air is found from equation 17, using column 6 and 10 summations:

$$T = \frac{2\pi}{3.567} \sqrt{\frac{3 \times 110.62}{525000 \times 32.2} \times 113.32} = 0.083 \text{ sec.}$$

The effective width of the riser, B_e, is determined using the summation of column 11 of table 3 as:

$$B_e = \frac{115.89}{27.0} = 4.29 \text{ ft}$$

By equation 18 the upper limit in air is:

$$T = 0.05 \times 29.17/\sqrt{4.29} = 0.70 \text{ sec.}$$

From equation 48:

$$\beta_e = \frac{4.29}{28.92} = 0.148$$

Hence from equation 46, the fundamental period of vibration of the fully submerged projecting portion of the riser is:

$$T_{wf} = (1.46 - 0.77 \times 0.148 + 0.70 \times \overline{0.148}^2 \times 0.083$$

$$= 1.36 \times 0.083 = 0.113 \text{ sec}$$

β_i is computed from equation 44 and given in column 12. The base values ζ_i of the added mass ellipses are computed from equation 42 and given in column 13. The ellipse ordinate values, r, are computed from equation 35 and given in column 14. The ellipse abscissa values, z_i, are computed from equation 41 and given in column 15. The added weight per ft for each segment is computed from equation 40 and given in column 16. The total added weight per segment is computed from equation 45 and given in column 17. The three elements of segment weight are summed from columns 6, 17, and 19 and are given in column 20.

The base shear coefficient is:

$$C = 0.05/(0.113)^{1/3} = 0.103$$

which exceeds 0.10, so use:

$$C = 0.10$$

The total base shear from equation 53, is:

$$V_o = (0.50 \times 1.50 \times 2.0 \times 0.10 \times 188.70)(1.0 \times 1.0)$$

$$= 0.150 \times 188.70 = 28.31 \text{ kips}$$

The top lateral force from equation 21, is:

$$F_t = 0.004\left(\frac{28.92}{4.29}\right)^2 \times 28.31 = 0.18 \times 28.31$$

since this is more than $0.15V_o$, use:

$$F_t = 0.15 \times 28.31 = 4.25 \text{ kips}$$

The lateral force on any segment is found from equation 22 and the values are given in column 22. For example for segment 6:

$$F_6 = (28.31 - 4.25)\,\frac{253.50}{2804.63} = 2.17 \text{ kips}$$

The shear and moment values at any level x are found by equations 23 and 27 or may be determined directly from statics. Note that the statical

Table 3. Partially embedded riser, projecting portion analysis, in-water condition, fully submerged.

Segment i	h_i (ft)	B_i (ft)	L_i (ft)	w_i (klf)	W_i(air) (kips)	H_i (ft)	H_w-H_i (ft)	I_i (ft^4)	$(H_w-H_i)^2\frac{h_i}{I}$ (ft^{-1})	$B_i h_i$ (ft^2)	β_i -	ζ_i -	r -
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
Top Lateral Force	-	-	-	-	-	-	-	-	-	-	-	-	-
13	0.67	-	9.17	19.48	12.99	28.58	-	-	-	-	-	-	-
12s	1.92	4.17	9.17	-	-	27.97	-	-	-	-	0.144	0.111	0.967
12	1.25	-	9.17	3.54	4.43	27.63	-	-	-	-	-	-	-
11	0.50	-	9.17	2.50	1.25	26.75	-	-	-	-	-	-	-
10	5.00	-	9.17	1.25	6.25	24.83	-	-	-	-	-	-	-
9	7.00	4.17	9.17	2.92	20.46	23.50	3.50	45.6	1.80	29.19	0.144	0.111	0.813
8	5.00	4.17	9.17	2.92	14.62	17.50	9.50	45.6	9.90	20.85	0.144	0.111	0.605
7	5.00	4.17	9.17	2.92	14.62	12.50	14.50	45.6	23.05	20.85	0.144	0.111	0.432
6	5.00	4.5	9.5	3.60	18.00	7.50	19.50	62.4	30.47	22.50	0.156	0.120	0.259
5	5.00	4.5	9.5	3.60	18.00	2.50	24.50	62.4	48.10	22.50	0.156	0.120	0.086
Σ					110.62				113.32	115.89			

Segment i	z_i	w_{ai} (klf)	W_{ai} (kips)	w_{wi} (klf)	W_{wi} (kips)	W_i (kips)	W_iH_i (kip-ft)	F_j kips	H_x (ft)	V_x (kips)	M_s (ft-kips)	J_x -	M_x (ft-kips)
	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)	(26)	(27)
Top Lateral Force	-	-	-	-	-	-	-	4.25	28.92	4.25	0.0	1.0	0.0
13	-	-	-	-	-	12.99	371.25	3.18	28.25	7.43	3.9	↓	3.9
12s	0.028	0.46	0.88	1.17	2.25	3.13	87.55	0.75	27.00	-	-	↓	-
12	-	-	-	-	-	4.43	122.40	1.05	27.00	9.23	14.6	↓	14.6
11	-	-	-	-	-	1.25	33.44	0.29	26.50	*9.52	*19.2	↓	*19.2
10	-	-	-	-	-	6.25	155.19	1.33	21.50	*10.85	*71.3	↓	*71.3
9	0.065	1.08	7.56	1.17	8.19	36.21	850.94	7.30	20.00	18.15	113.	↓	113.
8	0.088	1.46	7.30	1.17	5.85	27.77	485.98	4.18	15.00	22.33	214.	↓	214.
7	0.100	1.65	8.25	1.17	5.85	28.72	359.00	3.08	10.00	25.41	334.	↓	334.
6	0.116	1.99	9.95	1.17	5.85	33.80	253.50	2.17	5.00	27.58	477.	↓	477.
5	0.120	2.06	10.30	1.17	5.85	34.15	85.38	0.73	0.00	28.31	617.	1.0	617.
Σ						188.70	2804.38	28.31					

*The values are influenced by segment selection near top of riser; values above and below are valid.

moment reduction factor computed from equation 26, is:

$$J_o = 0.6/(0.113)^{1/3} = 1.24$$

which is greater than 1.0 so all:

$$J_x = 1.00$$

and hence, all:

$$M_x = M_s$$

Analysis of embedded portion of riser.--The analysis of the riser over the effective embedment depth proceeds by statics using the free body diagram of sketch (d) of figure 8. Pertinent values for this analysis are given in figure 15.

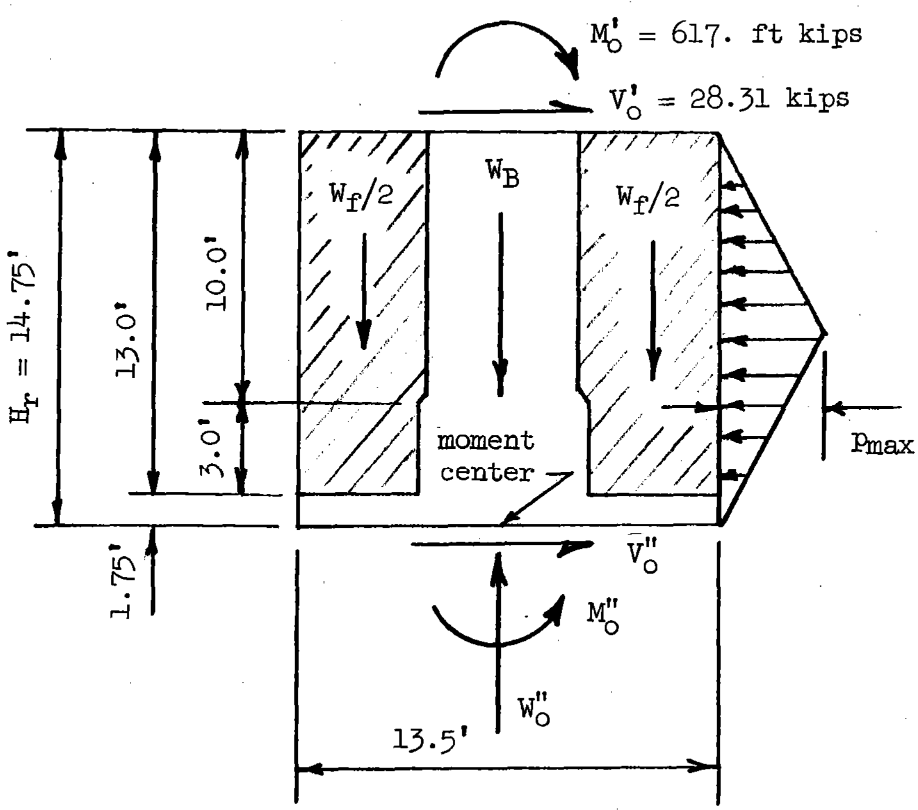


Figure 15. Effective embedded portion of riser.

The buoyant riser weight, W_B, was determined in the previous fully submerged analysis, as:

$$W_B = 123.96 \text{ kips}$$

The weight of the full 20 ft height of constructed fill, W_f, (buoyant weight here since the fill is submerged) is given approximately by:

$$\frac{1}{2} W_f = 10.5 \times \frac{67.6}{1000.} \times (4.0 \times 3.0 + 4.25 \times 10.0 + 4.5 \times 7.0)$$

or:

$$W_f = 122.09 \text{ kips}$$

Hence the contact bearing reaction is:

$$W_o'' = W_B + W_f = 123.96 + 122.09 = 246.05 \text{ kips}$$

The limiting value of p_{max} is computed from equation 52, as:

$$p_{max} = (2.5 - 0.40) \times 67.6 \times 14.75/2$$

$$= 1047. \text{ psf}$$

The weighted width of the horizontal pressure diagram is:

$$L_e = (10.5 \times 4.75 + 10.0 \times 10.0)/14.75 = 10.16 \text{ ft}$$

The value of p_{max} that will make M_o'' equal zero is found by summing moments about the midpoint of the riser footing. Thus:

$$M_o'' = 0. = 617. + 28.31 \times 14.75 - \frac{1}{2} p_{max} \times 14.75 \times \frac{14.75}{2} \times 10.16$$

from which:

$$p_{max} = 1.87 \text{ ksf}$$

This exceeds the limiting value, therefore p_{max} is set to its limiting value and the resulting M_o'' is found from statics as:

$$M_o'' = 617. + 28.31 \times 14.75 - \frac{1}{2} \times 1.047 \times \overline{\frac{14.75}{2}}^{2} \times 10.16$$

$$= 456. \text{ ft kips}$$

Again from statics:

$$V_o'' = \frac{1}{2} \times 1.047 \times 14.75 \times 10.16 - 28.31$$

$$= 50.14 \text{ kips}$$

Note that both V_o' and V_o'' oppose the induced resultant lateral earth pressures. Therefore the maximum vertical bending moment occurs within the embedded portion since shear passes through zero within this distance.

The eccentricity of the effective weight of riser plus submerged fill, W_o'', is:

$$e = \frac{456.}{246.05} = 1.85 \text{ ft}$$

this is less than:

$$\frac{B}{6} = \frac{13.5}{6} = 2.25 \text{ ft}$$

so that the bearing pressure diagram is trapezoidal. The maximum contact bearing pressure is given by:

$$p = \frac{W_o''}{LB} \left(1 + \frac{6e}{B}\right) \tag{70}$$

or: $$p = \frac{246.05}{10.5 \times 13.5}\left(1 + \frac{6 \times 1.85}{13.5}\right)$$

$$= 3.16 \text{ ksf} = 3160 \text{ psf}$$

Therefore, if the allowable contact bearing pressure for saturated conditions is:

$$\frac{4}{3} \times 2000 = 2670 \text{ psf}$$

the foundation is overstressed. However, if the allowable bearing includes an allowance for fill weight and hence is:

$$\frac{4}{3}(2000) + 67.6 \times 21.75 = 4137. \text{ psf}$$

the foundation bearing is adequate.

The safety factor against overturning need not be computed here since bearing is everywhere compressive. However, its value, assuming that the resultant lateral earth pressures contribute to the resisting moment, computes as:

$$SF_o = \frac{(246.05 \times 13.5/2 + 1.047 \times \overline{14.75}^2 \times 10.16/4)}{(617. + 28.31 \times 14.75)}$$

$$= \frac{1660.8 + 578.6}{1034.57} = \frac{2239.4}{1034.57} = 2.16$$

An alternate, though not desirable formulation is possible. If it is assumed that the resultant lateral earth pressures serve to reduce the overturning moment, the safety factor computes as:

$$SF_o = \frac{(246.05 \times 13.5/2\,)}{(617. + 28.31 \times 14.75 - 1.047 \times \overline{14.75}^2 \times 10.16/4)}$$

$$= \frac{1660.8}{1034.57 - 578.6} = \frac{1660.8}{456.} = 3.64$$

Stability of the riser depends on the base being able to develop the 50.14 kips shearing resistance. The safety factor is

$$SF_s = \frac{246.05 \times 0.35}{50.14} = 1.72$$

which is adequate.

With respect to horizontal bending in the embedded portion of the riser, maximum lateral earth pressures occur at $H_r/2 = 14.75/2 = 7.375$ ft above the base. The active pressure on the sidewall opposite the displacement side is $0.40 \times 67.6 \times 14.75/2 = 199$ psf. The pressure on the displacement side is $p_{max} + 199.$ or $1047. + 199. = 1246$ psf. Simultaneously with these earthquake induced pressures, the net inward loading due to pipe flow occurs.

The maximum value for standard risers, from Technical Release No. 30, is 3 x 0.96 x 62.4 = 180 psf. The inward loadings on the riser walls at 7.375 ft above the base are thus: 1246 + 180 = 1426. psf on one side-wall, 199 + 180 = 379 psf on the other sidewall, and 180 psf on the end-walls. An indeterminate analysis of these inward loadings is required to accurately determine the resulting stress conditions. Note that the 1426 psf loading by itself would produce far-side corner moments with tension on the insides of the corners. The loadings on the endwalls and the other sidewall would reduce the magnitude but probably not change the sense of these moments. Hence corner steel as shown in figure 11, sketch (g) may be indicated over some middle distance of the effective embedment depth.

Comparison of the results between the last two situations, that is, two fully submerged risers - one as a free-standing tower structure and one as a partially embedded structure, demonstrates the effect and value of partially embedding the riser where it is practical to do so.

REFERENCES

References are listed for those who wish to pursue the subject further. The list is by no means exhaustive.

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APPENDIX A

SEISMIC ANALYSIS OF RISERS

Computation of Riser Fundamental Periods of Vibration

In-air riser taken as nonprismatic cantilever beam. Periods about both principal axes. Application of equation 17.

Assumptions:

Riser inside plan dimensions, D x 3D

4000 psi concrete

E = 525,000. ksf

Figure A.1. Computer program.

Figure A.2. Input/output example.

Table A.1. Standard Covered Risers, ES-30DD, risers in reservoir.

Table A.2. Standard Open Risers, ES-31DD, risers in reservoir.

Table A.3. Standard Baffle Risers, ES-32DD, risers in reservoir.

Table A.4. Standard Covered Risers, ES-30DD, risers in embankment.

Table A.5. Standard Open Risers, ES-31DD, risers in embankment.

Table A.6. Standard Baffle Risers, ES-32DD, risers in embankment.

Herein:

TE ≡ Fundamental period, bending axis normal to plane of endwall.

TS ≡ Fundamental period, bending axis normal to plane of sidewall.

```
**** TSO FOREGROUND HARDCOPY ****
DSNAME=SCS05.PERIODS.FORT

C-----
C-----COMPUTATION OF RISER FUNDAMENTAL PERIODS OF VIBRATION,
C-----NONPRISMATIC CANTILEVER BEAM, TR-XX, EQUATION (17).
C-----
C-----              PREPARED BY
C-----              ES ALLING AND JF ROBISON
C-----              DESIGN UNIT, NATIONAL ENGINEERING STAFF
C-----              SOIL CONSERVATION SERVICE
C-----
C-----DECK DATE -- JANUARY 12, 1982
C-----
      DIMENSION TFT(10),HS(10),HB(10),HD(10),HDSQH(10),SW(10),EW(10),
     1RIS(10),STERM(10),RIE(10),ETERM(10),NUMBER(10)
      NREAD  = 5
      NRITE  = 6
      R12    = 1.0/12.0
      PI     = 3.14159
      E      = 525000.
      G      = 32.2
      TPART  = (2.*PI/3.567)*(3./(E*G))**0.5
      WRITE (NRITE,10)
   10 FORMAT (' '/14X,'COMPUTATION OF RISER FUNDAMENTAL PERIODS OF VIBRA
     1TION'/16X,'NONPRISMATIC CANTILEVER BEAM, TR-XX, EQUATION(17)')
  100 READ (NREAD,1,END=10000) NUMBER,HW,DIAM,VOL,SW(1),EW(1),RM
    1 FORMAT (10A1,6F10.3)
      WRITE (NRITE,14)
   14 FORMAT (' '/8X,'NUMBER',7X,'HW',8X,'DIAM',6X,'VOL',7X,'SW(1)',5X,'
     1EW(1)',6X,'SEGS')
      WRITE (NRITE,2) NUMBER,HW,DIAM,VOL,SW(1),EW(1),RM
    2 FORMAT (' ',5X,10A1,6F10.3)
      D      = DIAM*R12
      WT     = VOL*0.150*27.
      M      = RM
      WRITE (NRITE,15)
   15 FORMAT (' '/11X,'T(J)',4X,'HSML(J)',3X,'HBIG(J)')
      DO 200 J=1,M
      READ (NREAD,3) TFT(J),HS(J)
    3 FORMAT (2F10.3)
      IF (J.GT.1) GO TO 101
      HB(1)  = HS(1)/2.
      GO TO 102
  101 HB(J)  = HB(J-1)+(HS(J-1)+HS(J))/2.
  102 WRITE (NRITE,4) TFT(J),HS(J),HB(J)
    4 FORMAT (' ',5X,3F10.3)
      TFT(J) = TFT(J)*R12
  200 CONTINUE
      WRITE (NRITE,16)
   16 FORMAT (' '/11X,'SW(J)',5X,'EW(J)',4X,'IS(J)',4X,'STERM(J)',3X,
     1'IE(J)',4X,'ETERM(J)')
      SUMST  = 0.
      SUMET  = 0.
      RISD   = R12*D*(3.*D)**3.0
      RIED   = R12*(3.*D)*D**3.0
      DO 300 J=1,M
      HD(J)  = HW-HB(J)
      HDSQH(J) = HD(J)**2.*HS(J)
      IF (J.GT.1) GO TO 201
      RIS(1) = R12*EW(1)*SW(1)**3.0
      STERM(1) = HDSQH(1)/RIS(1)
      RIE(1)   = R12*SW(1)*EW(1)**3.0
      ETERM(1) = HDSQH(1)/RIE(1)
      GO TO 202
  201 SW(J)  = 3.*D+2.*TFT(J)
      EW(J)  =  D+2.*TFT(J)
      RIS(J)  = R12*EW(J)*SW(J)**3.0-RISD
      STERM(J) = HDSQH(J)/RIS(J)
      RIE(J) = R12*SW(J)*EW(J)**3.0-RIED
      ETERM(J) = HDSQH(J)/RIE(J)
  202 SUMST  = SUMST+STERM(J)
      SUMET  =  SUMET+ETERM(J)
      WRITE (NRITE,12) SW(J),EW(J),RIS(J),STERM(J),RIE(J),ETERM(J)
   12 FORMAT (' ',5X,6F10.3)
  300 CONTINUE
      TS     = TPART*(WT*SUMST)**0.5
      TE     = TPART*(WT*SUMET)**0.5
      WRITE (NRITE,13)
   13 FORMAT (' '/8X,'NUMBER',7X,'WT',18X,'SUMST',7X,'TS',6X,'SUMET',7X,
     1 'TE')
      WRITE (NRITE,5) NUMBER,WT,SUMST,TS,SUMET,TE
    5 FORMAT (' ',5X,10A1,F10.3,10X,4F10.3/5X,72('='))
      GO TO 100
10000 WRITE (NRITE,6)
    6 FORMAT (' '/5X,'END OF INPUT DATA, JOAN FOR ESA')
      STOP
      END
```

Figure A1. Computer Program.

```
**** TSO FOREGROUND HARDCOPY ****
DSNAME=SCS05.PERIODS.DATA

JOANFORESA    41.75      30.        52.41      10.5       13.5       9.0
   0.          1.75
  18.          3.0
  15.          5.0
  15.          5.0
  12.          5.0
  12.          5.0
  10.          5.0
  10.          5.0
  10.          7.0
```

(a) Input for TR example riser.

```
1      //SCS5FORT JOB (            ,RJ118,,,,,,,00),'DESIGN UNIT - 67377',        JOB 3529
       //   PRTY=3,
       //   CLASS=C,TIME=(,20),MSGLEVEL=(1,1)
       ***ROUTE   PRINT RMT146
2      //JOBLIB DD DSN=SCS05.PERIODS.LOAD(TEMPNAME),DISP=SHR
3      //   EXEC PGM=TEMPNAME
4      //GO.FT06F001 DD SYSOUT=A
5      //GO.FT08F001 DD SYSOUT=A
6      //GO.FT05F001 DD DSN=SCS05.PERIODS.DATA,DISP=SHR
```

COMPUTATION OF RISER FUNDAMENTAL PERIODS OF VIBRATION
NONPRISMATIC CANTILEVER BEAM, TR-XX, EQUATION(17)

NUMBER	HW	DIAM	VOL	SW(1)	EW(1)	SEGS
JOANFORESA	41.750	30.000	52.410	10.500	13.500	9.000

T(J)	HSML(J)	HBIG(J)
0.0	1.750	0.875
18.000	3.000	3.250
15.000	5.000	7.250
15.000	5.000	12.250
12.000	5.000	17.250
12.000	5.000	22.250
10.000	5.000	27.250
10.000	5.000	32.250
10.000	7.000	38.250

SW(J)	EW(J)	IS(J)	STERM(J)	IE(J)	ETERM(J)
10.500	13.500	1302.324	2.245	2152.828	1.358
10.500	5.500	442.686	10.045	135.812	32.742
10.000	5.000	328.775	18.101	94.401	63.042
10.000	5.000	328.775	13.235	94.401	46.093
9.500	4.500	233.624	12.846	62.375	48.116
9.500	4.500	233.624	8.138	62.375	30.481
9.167	4.167	179.559	5.855	45.492	23.108
9.167	4.167	179.559	2.513	45.492	9.919
9.167	4.167	179.559	0.478	45.492	1.885

NUMBER	WT	SUMST	TS	SUMET	TE
JOANFORESA	212.260	73.456	0.093	256.745	0.173

==

END OF INPUT DATA, JOAN FOR ESA

(b) Output for TR example riser.

Figure A2. Input/output example.

Table A1. Standard Covered Risers, ES-30DD, risers in reservoir.

TS & TE IN SEC.	3024		3030		3036		3042		3048	
	TS	TE	TS	TE	TS	TE	TS	TE	TS	TE
4035R	.113	.205	.092	.173	.076	.143	.065	.121	.058	.108
4030R	.121	.224	.093	.175	.078	.145	.065	.121	.059	.112
4025R	.121	.226	.094	.176	.078	.147	.068	.130	.059	.113
4020R	.123	.229	.094	.177	.083	.159	.068	.132	.059	.113
3530R	.091	.169	.071	.134	.059	.109	.052	.098	.046	.088
3525R	.091	.169	.071	.134	.062	.118	.052	.099	.046	.088
3520R	.093	.173	.072	.135	.062	.119	.052	.100	.048	.093
3515R	.101	.190	.077	.148	.063	.120	.055	.107	.048	.093
3025R	.069	.128	.056	.107	.046	.087	.039	.074	.036	.070
3020R	.073	.137	.056	.107	.046	.088	.040	.078	.036	.070
3015R	.073	.138	.057	.110	.047	.089	.041	.080	.037	.072
3010R	.075	.142	.060	.118	.049	.096	.041	.081	.038	.076
2520R	.052	.098	.040	.077	.035	.067	.030	.057	.027	.054
2515R	.052	.098	.042	.082	.035	.067	.030	.057	.027	.053
2510R	.053	.101	.043	.083	.036	.070	.031	.060	.028	.056
2505R	.053	.101	.044	.086	.037	.074	.032	.066	.029	.059
2015R	.033	.063	.028	.055	.025	.048	.021	.043	.019	.037
2010R	.033	.063	.028	.055	.025	.049	.021	.043	.019	.039
2005R	.034	.064	.028	.055	.026	.051	.022	.045	.019	.040
1510R	.020	.037	.017	.033	.015	.029	--	--	--	--
1505R	.020	.037	.017	.032	.015	.029	--	--	--	--

Table A2. Standard Open Risers, ES-31DD, risers in reservoir.

TS & TE IN SEC.	3124		3130		3136		3142		3148	
	TS	TE	TS	TE	TS	TE	TS	TE	TS	TE
3535R	.084	.152	.068	.127	.057	.105	.049	.090	.044	.081
3030R	.065	.120	.050	.094	.042	.078	.037	.070	.034	.063
2525R	.047	.086	.037	.071	.031	.058	.027	.051	.025	.048
2020R	.032	.060	.025	.047	.022	.041	.019	.036	.017	.034
1515R	.018	.034	.015	.029	.013	.026	.012	.024	.010	.020
1010R	.009	.016	.007	.014	.007	.013	--	--	--	--

Table A3. Standard Baffle Risers, ES-32DD, risers in reservoir.

TS & TE IN SEC.	3224		3230		3236		3242		3248	
	TS	TE	TS	TE	TS	TE	TS	TE	TS	TE
3535R	.087	.158	.071	.132	.060	.111	.051	.094	.045	.084
3030R	.068	.126	.053	.099	.045	.083	.039	.074	.035	.067
2525R	.050	.091	.040	.075	.033	.062	.029	.054	.026	.051
2020R	.035	.066	.027	.051	.024	.045	.020	.039	.019	.037
1515R	.020	.038	.017	.033	.015	.030	.014	.027	.011	.023
1010R	.010	.018	.008	.016	.008	.015	--	--	--	--

Table A4. Standard Covered Risers, ES-30DD, risers in embankment.

TS & TE IN SEC.	3024		3030		3036		3042		3048	
	TS	TE	TS	TE	TS	TE	TS	TE	TS	TE
4035E	.131	.242	.105	.199	.084	.159	.068	.130	.059	.111
4030E	.129	.243	.097	.184	.081	.153	.066	.125	.059	.114
4025E	.119	.223	.094	.178	.078	.148	.068	.131	.059	.113
4020E	.114	.214	.092	.174	.080	.154	.067	.130	.059	.113
3530E	.101	.190	.076	.144	.062	.117	.053	.102	.047	.089
3525E	.093	.174	.073	.138	.063	.122	.053	.101	.046	.088
3520E	.091	.172	.071	.134	.062	.119	.052	.100	.047	.092
3515E	.093	.177	.074	.143	.062	.118	.064	.106	.047	.091
3025E	.071	.134	.058	.112	.047	.090	.040	.076	.036	.071
3020E	.072	.138	.057	.109	.046	.089	.040	.079	.036	.070
3015E	.069	.132	.056	.108	.046	.088	.040	.080	.036	.072
3010E	.070	.133	.059	.114	.048	.094	.040	.080	.037	.074
2520E	.053	.102	.041	.079	.035	.068	.029	.058	.027	.054
2515E	.050	.095	.042	.081	.034	.068	.029	.057	.027	.053
2510E	.050	.094	.042	.081	.035	.069	.030	.060	.027	.055
2505E	.050	.095	.093	.084	.037	.073	.032	.065	.028	.057
2015E	.033	.063	.029	.056	.024	.048	.021	.043	.019	.037
2010E	.033	.063	.028	.055	.024	.048	.021	.043	.020	.041
2005E	.033	.063	.028	.055	.025	.050	.022	.045	.019	.039
1510E	.020	.037	.017	.033	.015	.030	--	--	--	--
1505E	.020	.037	.017	.033	.015	.030	--	--	--	--

Table A5. Standard Open Risers, ES31-DD, risers in embankment.

TS & TE IN SEC.	3124		3130		3136		3142		3148	
	TS	TE	TS	TE	TS	TE	TS	TE	TS	TE
3535E	.100	.182	.079	.149	.063	.118	.051	.097	.045	.084
3030E	.073	.137	.054	.102	.045	.084	.038	.073	.034	.064
2525E	.049	.091	.039	.074	.032	.060	.027	.052	.025	.048
2020E	.033	.064	.025	.048	.021	.042	.019	.036	.017	.034
1515E	.018	.034	.015	.030	.013	.026	.012	.023	.010	.020
1010E	.008	.016	.007	.014	.006	.013	--	--	--	--

Table A6. Standard Baffle Risers, ES-32DD, risers in embankment.

TS & TE IN SEC.	3224		3230		3236		3242		3248	
	TS	TE	TS	TE	TS	TE	TS	TE	TS	TE
3535E	.103	.188	.082	.154	.066	.123	.054	.101	.046	.087
3030E	.077	.143	.057	.107	.047	.089	.041	.077	.036	.068
2525E	.052	.096	.041	.079	.034	.064	.029	.055	.026	.051
2020E	.036	.069	.027	.052	.024	.046	.020	.039	.019	.037
1515E	.020	.038	.017	.033	.015	.030	.014	.027	.011	.023
1010E	.010	.018	.008	.016	.008	.015	--	--	--	--

APPENDIX B

SEISMIC ANALYSIS OF RISERS

Hints for Preliminary Investigations

Risers respond to earthquake ground motion. The response includes the generation of displacements and accelerations within the structure. Lateral forces are induced. These in turn produce shears and moments with resulting concrete and reinforcing steel stresses. External considerations include sliding, overturning, and bearing.

The severity of the response depends on: the configuration of the structure, the material properties of the structure, the site profile and properties, and the site location with respect to potential earthquakes. The parameters contained in equation 53 are indices of these factors. Often the response will be slight and detailed consideration of the earthquake effects is unwarranted.

Several quick approximations are possible which can be used to determine if more refined analyses are advisable. Some are given below. More will be discovered as experience with seismic analyses is gained.

Base Shear Coefficient

Rather than computing the fundamental period of vibration, take the base shear coefficient, C, at its maximum value. Thus say:

$$C = 0.10.$$

Distribution of Lateral Forces

Rather than computing the concentrated force applied at the top of the riser and the lateral forces applied to each segment, assume the total lateral load (equal to the total base shear, V_o) is concentrated and applied at the top of the riser.

Statical Moment Reduction Factors

Rather than computing statical moment reduction factors, take them at maximum value. Thus say:

$$J_o = J_x = 1.0.$$

Determination of Added Mass

Rather than computing the added mass for each segment within the submerged height, H_h, make one computation for the entire submerged height. Thus take:

$$r = 0.50$$

$$\beta = B/H_s \quad \text{where B is width of riser at } H_h/2$$

then:

$$\zeta = 0.8\beta - 0.2\beta^2 \quad \text{assuming } \beta \leq 2$$

$$z = \zeta\sqrt{.75} = 0.87\zeta$$

and:

$$w_a = z\gamma_w H_h L \quad \text{where L is value at } H_h/2$$

so:

$$W_a = w_a H_h$$

Soil-Structure Interaction Factor

Take the interaction factor at its maximum value and do not iterate analyses. Thus say:

$$I = 1.0.$$

APPENDIX C

SEISMIC ANALYSIS OF RISERS

Some English to Metric Conversion Factors

To Convert from		to	Multiply by
in		mm	25.4
in		cm	2.54
ft		m	0.3048
in^2		cm^2	6.4516
ft^2		m^2	0.09290
in^3		cm^3	16.3871
ft^3		m^3	0.02832
in^4		cm^4	41.6231
ft^4		m^4	0.008631
ft/s	(fps)	m/s	0.3048
ft/s^2		m/s^2	0.3048
ft^3/s	(cfs)	m^3/s	0.02832
lb	(mass, weight)	kg	0.4536
lb/ft^3	(pcf)	kg/m^3	16.0185
lb	(force)	N	4.4482
lb/in^2	(psi)	kPa	6.8948
lb/ft^2	(psf)	kPa	0.04788

U.S. GOVERNMENT PRINTING OFFICE 1982-0- 522-010/3707